PROCEEDINGS

OF THE

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OF

CIVIL ENGINEERS

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

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This Society is not responsible for any statement made or opinion expressed in its publications.

SOCIETY AFFAIRS

CONTENTS

MINUTES OF MEETINGS
OF THE SOCIETY

November 16th, 1921.—The meeting was called to order at 2.15 P. M.; President George S. Webster in the chair; Elbert M. Chandler, Acting Secretary; and present, also, 118 members and guests.

The afternoon was devoted to an informal discussion of the subject, "Stream Pollution and Sewage Disposal". The discussion was opened by George T. Hammond, M. Am. Soc. C. E., who addressed the meeting on "Tanks and Fine Screens for Treating Sewage". Mr. Hammond was followed by Kenneth Allen, M. Am. Soc. C. E., who discussed the "Pollution of Tidal Harbors by Sewage", illustrating his remarks with lantern slides. John F. Skinner, M. Am. C. E., spoke on "Storm-Water Treatment", and W. L. Stevenson, M. Am. Soc. C. E., explained the "Policies of the Pennsylvania Department of Health". The subject discussed by Earle B. Phelps, Affiliate, Am. Soc. C. E., was "Stream Pollution and Its Control", and T. Chalkley Hatton, M. Am.

Soc. C. E., described the "Deposition of Sludges from Sewage Disposal Plants". A discussion on "The Dilution Factor", by Langdon Pearse, M. Am. Soc. C. E., was presented by Dr. F. W. Mohlman, and W. H. Dittoe, M. Am. Soc. C. E., spoke on "Prevention of Misuse of Sewers".

The addresses were followed by oral discussion of the general subject by Messrs. Harrison P. Eddy, Kenneth Allen, J. F. Jackson, Glenn D. Holmes, F. A. Dallyn, W. F. Wells, Rudolph Hering, Alexander Potter, Edward S. Rankin, C. M. Baker, P. H. Norcross, and S. John Scacciaferro.

Adjourned.

November 16th, 1921.—The regular business meeting of the Society was called to order at 8.10 P. M.; President George S. Webster in the chair; Elbert M. Chandler, Acting Secretary; and present, also, 95 members and guests.

The minutes of the meeting of October 5th, 1921, were approved as printed in the *Proceedings* for October, 1921.

The Acting Secretary announced the election of the following candidates on October 10th, 1921:

As Members

Francis De Schauensee, New York City
Alfred Fellheimer, New York City
Charles Thurston Fisher, Memphis, Tenn.
Lyman Griswold, Portland, Ore.
Francis Hathaway Hardy, Washington, D. C.
Raymond Dudley Hoyt, Portland, Ore.
Edward Neele Johnston, Wilmington, Del.
Henry Marvin Lilly, Beaufort, N. C.
Neil McIntyre Loney, New York City
Peter Alexander McLeod, Webster Groves, Mo.
Joseph Warren Parker, Boston, Mass.
Clair Leverett Peck, Los Angeles, Cal.
Walter Putnam, Pasadena, Cal.
Charles Germane Richardson, Providence, R. I.
Perry Thomas Simons, Washington, D. C.
Raphael Joseph Smyth, New York City
Isaac Joshua Stander, New York City

As Associate Members

Arthur Quintin Adamson, Shanghai, China
Carl Toevs Baer, Dallas, Tex.
Julian Norman Ball, Omaha, Nebr.
Edwin John Brockmeyer, East St. Louis, Ill.
George Robert Burtner, Greenville, Tex.
Henry Bowers Campbell, Norfolk, Va.
Joseph Phillip Carey, Marks, Miss.
Neil McComas Cecil, Modesto, Cal.
Holton Cook, Dixon, Ky.

William Howard Corddry, Memphis, Tenn.
Theodore Cronyn, Plandome, N. Y.
Philip Jacob Endlich, Detroit, Mich.
Augustus Bernard Fleck, Woodhaven, N. Y.
William Edward Gabelman, Manila, Philippine Islands
Adrian John Gilardi, Cambridge, Mass.
James Raymond Green, Pittsburgh, Pa.
George Edwin Howe, Jamaica, N. Y.
Fred Kellam, Indianapolis, Ind.
Thomas Leon Spoore Landers, Edmundston, N. B., Canada
Morris Wooten Loving, Chicago, Ill.
Thomas Brandon Munroe, Cedartown, Ga.
Carrol Clifford Nicholls, University Place, Nebr.
Charles Robert Porter, Middlesbrough, England
Case Broderick Rafter, Washington, D. C.
Cody Sylvester Reagan, Dallas, Tex.
Fred White Sarvis, Harlan, Iowa
Sheldon Beardsley Shepard, Duluth, Minn.
Paul Revere Shields, Hazard, Ky.
Harry Augustus Shuptrine, Detroit, Mich
Carl Waldemar Smedberg, St. Paul, N. C.
Ralph Jerome Smith, Rochester, N. Y.
George Henry Van Cott, Glen Head, N. Y.
Oliver William Van Petten, Ashland, Ky.
Adolph Gottig Weber, Berkeley, Cal.

As Juniors

Cornelius Alfred Boyle, New York City
Rex Lenoi Brown, Urbana, Ill.
Linden Van Horn Fisher, Roanoke, Va.
Samuel Robert Goldman, Nebo, N. C.
Walter Edmund Grasheim, New York City
Reinhold Bernhard Hansen, San Francisco, Cal.
Llewellyn Gilmore Haskell, Berkeley, Cal.
John Blackstock Hawley, Jr., Fort Worth, Tex.
Fred Hendershot, Chicago, Ill.
William John McGrath, New York City
Thomas Meloy, New York City
William Francis Rooney, New York City
Kenneth Ward Ross, New York City
Salvator John Scacciaferro, Clifton, N. J.
Keith Henry Swanholm, Boise, Idaho
Charles Oscar Thomas, Benton, Ark.
Guy Bennett Waite, Jr., New York City
Fayette Samuel Warner, Sunderland, Mass.
Lyman Dwight Wilbur, San Francisco, Cal.
Choong Wai Woo, Shanghai, China

The Acting Secretary announced the transfer of the following candidates on October 11th, 1921:

From Associate Member to Member

Porter Hugh Albright, Los Angeles, Cal.
Harland Bartholomew, St. Louis, Mo.
William Walter Bigelow, Boston, Mass.
William Dollison Faucette, Norfolk, Va.
Arthur Jenkins Ford, Phœnix, Ariz.
Paul Charles Gauger, St. Paul, Minn.
Richard Ambrose Hart, Salt Lake City, Utah
Earle Underwood Henry, Houston, Tex.
Robert Leslie Holmes, Dallas, Tex.
Ivan Edgar Houk, Dayton, Ohio
Frank Alvah Kittredge, Missoula, Mont.
Charles Abraham Lass, Birmingham, Ala.
Stanley Macomber, Massillon, Ohio
Roy Everett Miller, Seattle, Wash.
Olaf Otto, Savannah, Ga.
Guy Pinner, New York City
William Edward Rudolph, Fort Madison, Iowa
James Frederick Scrimshaw, Arlington, N. J.
Charles Willett Spooner, Grand Rapids, Mich.
Clarence McNaughton Steeves, Edmundston, N. B., Canada
Hans Von Unwerth, Kansas City, Mo.

From Junior to Associate Member

Franklin Harper Craddock, Centralia, Wash.
Charles William Doerr, Emsworth, Pa.
Ragsdale Pace, Fort Worth, Tex.
Walter Raymond Weber, Denver, Colo.

The Acting Secretary announced the following deaths:

Hiram Francis Mills, of Hingham, Mass., elected Honorary Member, November 30th, 1909; died October 4th, 1921.

William Edgar Baker, of New York City, elected Member, June 1st, 1898; died November 7th, 1921.

James Simpson Browne, of New Haven, Conn., elected Associate Member, October 4th, 1893; Member, July 1st, 1909; died October 22d, 1921.

Frederick William Cappelen, of Minneapolis, Minn., elected Member, April 3d, 1895; died October 16th, 1921.

Samuel Merrill Gray, of Providence, R. I., elected Member, May 15th, 1872; died November 6th, 1921.

Peter Conover Hains, of Washington, D. C., elected Member, April 2d, 1890; died November 7th, 1921.

Howard Carleton Holmes, of San Francisco, Cal., elected Member, November 4th, 1903; died October 30th, 1921.

Sir JOHN KENNEDY, of Montreal, Que., Canada, elected Member, September 1st, 1875; died October 25th, 1921.

PHILO SACKETT PERKINS, of Providence, R. I., elected Junior, March 5th, 1890; Associate Member, April 3d, 1895; Member, July 11th, 1921; died October 28th, 1921.

CHARLES WARD RAYMOND, of Sacramento, Cal., elected Junior, November 7th, 1877; Member April 7th, 1886; died October 27th, 1921.

GEORGE DUNCAN SNYDER, of Jersey Shore, Pa., elected Associate Member, November 6th, 1895; Member, September 4th, 1901; died October 21st, 1921.

GEORGE HERBERT WEBB, of Detroit, Mich., elected Member, February 1st, 1893; died November 3d, 1921.

DUDLEY CHIPLEY, of Columbus, Ga., elected Associate Member, September 11th, 1917; died August 20th, 1921.

SAMUEL ALEXANDER FORTER, of Springdale, Pa., elected Associate Member, October 1st, 1912; died August 3d, 1921.

GISLI GUDMUNDSSON, of Pittsburgh, Pa., elected Associate Member, January 3d, 1900; died July 19th, 1921.

HENRY HARVIE, of Toronto, Ont., Canada, elected Associate Member, May 12th, 1919; died October 14th, 1921.

RALPH EWART ROBSON, of Berkeley, Cal., elected Junior, October 3d, 1911; Associate Member, June 3d, 1915; died October 14th, 1921.

ANDREW FRANCIS ROSS, of Los Angeles, Cal., elected Associate Member, June 11th, 1917; died May 8th, 1921.

HUGO JULIUS SCHEUERMANN, of Albany, N. Y., elected Associate Member, December 7th, 1904; died July 20th, 1921.

A paper by L. L. Tribus, M. Am. Soc. C. E., entitled "Odors and Their Travel Habits", was presented by the author. Various phases of the subject were discussed, among which were the following: "Physiology and Government Control of Odors", by George C. Whipple, M. Am. Soc. C. E.; "Observations of Odors in Rhode Island", by Stephen DeM. Gage, Chemist and Sanitary Engineer, State Board of Health of Rhode Island; "Elimination of Odors Produced by Garbage Disposal Plants", by I. S. Osborn, M. Am. Soc. C. E.; "Some General Observations", by Rudolph Hering, M. Am. Soc. C. E., which was read by the Acting Secretary; "Some Interstate Odors", by Olin H. Landreth, M. Am. Soc. C. E.; and "My Studies of Odors", by Robert S. Weston, M. Am. Soc. C. E. The general subject was discussed orally by Messrs. F. A. Dallyn and Alexander Potter.

Adjourned.

November 17th, 1921.—The meeting was called to order at 8.00 P. M.; Vice-President Francis Lee Stuart in the chair; Elbert M. Chandler, Acting Secretary; and present, also, 128 members and guests.

A symposium on "Water Supply and Water Purification", was opened by George C. Whipple, M. Am. Soc. C. E., whose subject was "History of Water Purification". Professor Whipple was followed by Allen Hazen, M. Am. Soc. C. E., on "Recent Developments in Water Purification"; C. A. Emerson, Jr.,

M. Am. Soc. C. E., on "Interference with Water Filtration Plant Operation by Wastes from By-Product Coke Ovens and Gas-Works": C.-E. A. Winslow, Professor of Public Health, Yale University, on "Reduction in Typhoid Death Rate"; C. A. Holmquist, Director of the Division of Sanitation, New York Department of Health, on "The Effect of Water Purification and Improvements in Water Supplies on the Typhoid Death Rate in New York State"; Robert S. Weston, M. Am. Soc. C. E., on "Purification of Soft, Colored Waters"; and Samuel A. Greeley, M. Am. Soc. C. E., on "The Operation of Reservoirs for Water Supply". The general subject was discussed orally by Messrs. L. L. Tribus, P. H. Norcross, G. F. Catlett, John R. Baylis, H. Malcolm Pirnie, C. M. Baker, M. N. Baker, H. N. Bundesen, Robert Spurr Weston, and George W. Simons, Jr.

Adjourned.

December 7th, 1921.—The meeting was called to order at 9.15 A. M., at the Headquarters of the Society; President George S. Webster in the chair; and present, also, 102 members and guests.

The meeting was devoted to a discussion of Chapter IX, "Water-Proofing and Protective Treatment", and Chapter X, "Surface Finish", of the Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete.

Adjourned at 12.20 P. M., to meet at 2 P. M.

December 7th, 1921.—The meeting was called to order at 2. P. M.; Director Richard L. Humphrey in the chair; and present, also, 90 members and guests.

The topics for discussion at this meeting, in continuation of the discussion of the Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, were Chapters VI, "Depositing Concrete"; Chapter VII, "Forms"; and Chapter VIII, "Details of Construction", of the Report. These subjects were discussed in detail by those present.

Adjourned at 4.15 P. M., to meet at 8 P. M.

December 7th, 1921.—The regular business meeting of the Society was called to order at 8 P. M.; Director Richard L. Humphrey in the chair; Elbert M. Chandler, Acting Secretary; and present, also, 133 members and guests.

The Acting Secretary announced the election of the following candidates on November 21st, 1921:

As Members

Mikishi Abe, Urbana, Ill.
Richard Allison Backus, South Orange, N. J.
Frederick Kellogg Blue, San Francisco, Cal.
Robert Brown, Dallas, Tex.
Howard Everett Cousins, Arlington, Mass.
Frederick Loud Cranford, Brooklyn, N. Y.
Allston Dana, Manchester, Mass.
Melvin Lorenius Enger, Urbana, Ill.
James Harris Pliny Fisk, Walsenburg, Colo.

Gunni Jeppesen, Chicago, Ill.
James Aloysius McElroy, Bridgeport, Conn.
Frank Duff McEnteer, Clarksburg, W. Va.
William Kurtz Myers, Philadelphia, Pa.
Isaac Oesterblom, New York City
Fred Howland Peckham, Oklahoma, Okla.
Charles Henry Schaefer, Philadelphia, Pa.
Francis Raymond Izlar Sweeny, Anderson, S. C.
John Small Thompson, Detroit, Mich.
Charles Francis Wood, Rancagua, Chili

As Associate Members

Thomas Patton Adams, Fort Worth, Tex.
Wilson Turner Ballard, Baltimore, Md.
Max Arnold Berns, Chicago, Ill.
Harold Warren Burge, Cleveland, Ohio
Harold Thomas Burgess, Meriden, Conn.
Hunter Hancock Burton, Manassas, Va.
Max Lawrence Button, Medford, Mass.
Arthur Adam Augustine Carman, New York City
James Henderson Childs, Pasadena, Cal.
Miles Elliott Clark, Olympia, Wash.
Robert Waite Davis, Slidell, La.
Thomas Marsh Davis, Portland, Ore.
Leon Snell Dixon, Bangor, Me.
Leroy Burrows Fugitt, Kansas City, Mo.
John Alexander Fulkman, Lorain, Ohio
Frederick Louis Hargett, Mineral Springs, Ark.
Harvey Dixon Herbert, Carlisle, Pa.
Melvin Emerson Hiet, Nowata, Okla.
William Henry Hoff, North Arlington, N. J.
Dudley Frank Holtman, Washington, D. C.
Elmer Guy Hooper, New York City
Clarence James Huguet, Baton Rouge, La.
Robert George Jackson, Watertown, Mass.
Algot Ferdinand Johnson, Minneapolis, Minn.
Schuyler Sheldon Albert Keast, Philadelphia, Pa.
James Michael Kelly, Philadelphia, Pa.
Harry Robert Knight, Palatka, Fla.
Kenneth Earl Lancet, Indianapolis, Ind.
Herrick Johnson Lane, New Orleans, La.
Fred Justin Lewis, South Bethlehem, Pa.
Wallace Bright Livesay, Port Neches, Tex.
James Robert Losee, Detroit, Mich.
Charles William Lovell, Louisville, Ky.
Harry Maxwell Lukens, Los Angeles, Cal.
Bernard Reeves McBride, Columbus, Ind.

Joseph Muth McCoy, San Francisco, Cal.
Murray Holman Mellish, New York City
William Henry Mohr, Allentown, Pa.
Earle Bristol Moss, Niagara Falls, N. Y.
William Clayton Newell, Casper, Wyo.
Charles Harold Olmstead, Nashville, Tenn.
Edwin Herbert Pagenhart, Seattle, Wash.
Glenn Stuart Paxson, Prineville, Ore.
Willard Averell Pollard, Jr., Washington, D. C.
Henry Cyrus Porter, Kingsville, Tex.
Lawrence Elmer Raymond, High Point, N. C.
Peter Remsen, Washington, D. C.
Emory Douglas Roberts, Corvallis, Ore.
William Evans Rodgers, Louisville, Ky.
Otto Charles Rollman, Green Bay, Wis.
Cerf Rosenthal, San Francisco, Cal.
Frank Adam Schilling, Los Angeles, Cal.
Frederick Sievers Schwinn, Houston, Tex.
Ray Otto Shriver, Newton, Kans.
Gordon Pitman Smith, Redwood Falls, Minn.
Joseph Francis Still, St. Petersburg, Fla.
Ralph Penny Thompson, Coeburn, Va.
Edward Newton Todd, Oklahoma, Okla.
Fullton Espey Varner, Atlanta, Ga.
Chauncey J. Wiegner, Memphis, Mo.

As Affiliates

Ernest Lester Jones, Washington, D. C.
Edwin Ruthven Willard, Berkeley, Cal.
Vern Elwood Winell, Cleveland, Ohio

As Juniors

Paul Bauman, Phœnix, Ariz.
William Breuer, Philadelphia, Pa.
Harry Buttorff Dyer, Nashville, Tenn.
Edmund Madison Eastman, Atlanta, Ga.
Frank William Flittner, San Francisco, Cal.
Sydney Wood Garrison, Raleigh, N. C.
Earl Douglas Gornto, Norfolk, Va.
Ray Freeman Goudey, Los Angeles, Cal.
Barclay Adams Greene, Kansas City, Mo.
James Clarke Harding, Jr., Mt. Vernon, N. Y.
Louis Korn, Los Angeles, Cal.
Abraham Levin, New York City
Elbert Francis Lewis, Seattle, Wash.
Harry McGraw, Wellsburgh, W. Va.
Percy Ralph Robinson, Rochdale, England.

Carlton Jernegan Spear, New York City
Harold Beekman Storms, Mount Vernon, N. Y.
Charles Le Patourel Terry, Sydney, N. S. W., Australia
Robert Vawter, Logan, W. Va.
John Crossley Waddington, Sheffield, England
Oscar Widstrand, North Troy, N. Y.

The Acting Secretary announced the transfer of the following candidates on November 21st, 1921:

Transferred from Associate Member to Member

Edwin Learned Adams, Los Angeles, Cal.
Francis Neal Baldwin, Dallas, Tex.
Frederick Bayard Barshell, New York City
William Parker Butler, Nashville, Tenn.
Ray Sheppard Carberry, Imperial, Cal.
James Duncan Fowler, Dallas, Tex.
Charles Kirby Fox, Los Angeles, Cal.
Harry Otto Garman, Indianapolis, Ind.
Eugene Lucius Grunsky, San Francisco, Cal.
Charles Sumner Heidel, Helena, Mont.
Norman Haden Hill, Indianapolis, Ind.
Harry Griffith Hunter, Kansas City, Mo.
Oscar Henry Koch, Dallas, Tex.
Philip George Lang, Jr., Baltimore, Md.
Omar Evert Malsbury, Balboa Heights, Canal Zone, Panama
Ernest Lindley Myers, Dallas, Tex.
James Lafayette Parker, Columbia, S. C.
James Henry Payne, Los Angeles, Cal.
Leon Friend Peck, Hartford, Conn.
George Henry Preston, Bloomfield, N. J.
Guy Wickliffe Rice, Blythe, Cal.
Horatio Seymour, Santa Monica, Cal.
Lawrence Vinnedge Sheridan, Dallas, Tex.
Elwin Streeter Warner, Austin, Tex.
Maurice Anderson Webster, Philadelphia, Pa.
Jacob Paul Jones Williams, Flushing, N. Y.
Edwin Carlton Woodward, Fort Worth, Tex.
Stell Kay Young, Kenton, Ohio.

Transferred from Junior to Associate Member

Frank Palmer Arnold, Charleston, W. Va.
Harry Lewis Bayer, Luzerne, N. Y.
Floyd Carson Bedell, Pearl Harbor, Hawaii
Clyde Stanley Constant, Parsons, Kans.
James Perkins Ewin, New Orleans, La.
Howard Leslie Foster, Detroit, Mich.

James Bruce O'Brien, Harrisburg, Pa.
Emil Praeger, Brooklyn, N. Y.
Elbert Saunders Tillotson, New York City
Bernon Tisdale Woodle, New York City

The Acting Secretary announced the following deaths:

Sir Douglas Fox, of London, England, elected Corresponding Member, June 7th, 1871; Honorary Member, March 5th, 1901; died November 13th, 1921.

John Bealle Battle, of Mobile, Ala., elected Member, October 9th, 1917; died November 6th, 1921.

John Charles Quintus, of Buffalo, N. Y., elected Member, January 2d, 1889; died November 27th, 1921.

Thomas Delano Whistler, of London, England, elected Junior, March 5th, 1884; Member, May 2d, 1888; date of death unknown.

Richard Tuggle Goodwyn, Jr., of Athens, Ga., elected Associate Member, September 12th, 1921; died November 8th, 1921.

Charles Whiting Bradley, of Richmond, Va., elected Affiliate, June 19th, 1891; died January 14th, 1920.

James Francis Wrenn, of Norfolk, Va., elected Affiliate, September 6th, 1905; died November 2d, 1921.

George Lord Burrows, of Saginaw, Mich., elected Affiliate, February 3d, 1886; died November 9th, 1921.

The evening was devoted to a discussion of Chapter XI, "Design", of the Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, which subject was discussed by many of the members and guests present at the meeting.

Adjourned at 10.30 P. M., to meet at 9 A. M., on December 8th, 1921.

December 8th, 1921.—The meeting was called to order at 9 A. M.; Director Richard L. Humphrey in the chair; and present, also, about 110 members and guests.

The subject for discussion at this meeting was a continuation of the discussion of Chapter VIII, "Details of Construction", and Chapter XI, "Design", of the Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete. The discussion was informal, in which many members and guests present took part.

Adjourned at 12.15 P. M., to meet at 2 P. M.

December 8th, 1921.—The meeting was called to order at 2 P. M.; Director Richard L. Humphrey in the chair; and present, also, about 90 members and guests.

The meeting was devoted to a general discussion of the Tentative Specifications for Concrete and Reinforced Concrete as contained in the Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, and of the scope of the work of the Joint Committee.

Adjourned at 5 P. M., to meet at 8 P. M.

December 8th, 1921.—The meeting was called to order at 8 P. M.; Director Richard L. Humphrey in the chair; and present, also, about 135 members and guests.

The subjects discussed at this meeting were Chapter III, "Quality of Concrete"; Chapter IV, "Materials"; and Chapter V, "Proportioning and Mixing Concrete", of the Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete. These subjects were generally discussed by those present.

Adjourned.

OF THE BOARD OF DIRECTION

(Abstract)

November 21st, 1921.—The Board convened in regular meeting at 10.15 A. M., at the Headquarters of the Society; President George S. Webster in the chair; Elbert M. Chandler, Acting Secretary; and present, also, Messrs. Clark (came in at 11.00 A. M.), Cummings (came in at 10.55 A. M.), Curtis, Greene, Herschel, Hogan, Hovey, Hudson (came in at 12.00 M.), Humphrey, Hunt, Langthorn (came in at 11.35 A. M.), Pegram (came in at 11.05 A. M.), and Stuart (came in at 10.45 A. M.).

Ballots for membership were canvassed, resulting in the election of 19 Members, 60 Associate Members, 3 Affiliates, and 21 Juniors, and the transfer of 10 Juniors to the grade of Associate Member.

Twenty-eight Associate Members were transferred to the grade of Member.

A report from the Membership Committee was received and acted on.

Adjourned.

PROPOSED AMENDMENTS TO THE CONSTITUTION

Two sets of amendments to the recently Revised Constitution have been mailed to the membership. They will be in order for discussion at the Business Meeting to be held during the Annual Meeting in New York City, January 18th, 1922.

The first group reproduced herewith has been deemed necessary for the clarification of doubtful features pertaining to the Revised Constitution, by the Executive Committee of the Board of Direction, charged with the duty of acting "in all matters involving the operation of the Revised Constitution"; by members of the Board of Direction; and by members of the retired Committee on Referred Amendments; as appears from the signatures to it appended.

The second group reprinted herein, was presented by thirty or more Corporate Members of the Society, under the provisions of Article X, Section 3, of the Revised Constitution.

GROUP I

Amend Article II.—Membership:

Add Section below, and renumber present Section 9 Section 10:

"9.—The membership status of members of the Society in any grade, as it was immediately prior to November fifth, 1921, shall not be affected by amendments to the Constitution taking effect on that date, except that the Associates at that time shall thereafter be termed Affiliates."

Amend Article IV.—Dues:

Amend Section 3 by inserting after the first paragraph the following:

"Members residing outside of North America shall pay annual dues as follows: by Corporate Members, twenty dollars; Affiliates, fifteen dollars; Juniors, ten dollars."

Amend Article VII.—Nomination and Election of Officers:

Amend Section 4, by inserting in 15th line before the words "No vote", "In the first and second canvasses for Official Nominees", making the sentence read:

"In the first and second canvasses for Official Nominees no vote of a Corporate Member for a nominee for Vice-President resident outside of the zone in which the voter resides shall be counted; no vote of a Corporate Member for a nominee for Director resident outside of the district in which the voter resides shall be counted."

These amendments were signed by George S. Webster, A. M. Hunt, O. E. Hovey, Francis Lee Stuart, Clemens Herschel, Richard L. Humphrey, George H. Pegram, Robert A. Cummings, John P. Hogan, John C. Hoyt, J. S. Langthorn, C. C. Elwell, John W. Alvord, Willard Beahan, Baxter L. Brown, Clarence C. Brown, C. W. Hudson, P. Junkersfeld, Anson Marston, Paul H. Norcross, W. E. Rolfe, William Stoecker, Edward E. Wall, A. P. Davis, George Hallett Clark, Carleton Greene, A. S. Baldwin, J. F. Coleman, D. C. Henny, L. L. Hidinger, E. J. Schneider, A. N. Talbot, and George G. Anderson.

GROUP II

Amend Article VII.—Nomination and Election of Officers:

Amend Section 1, by striking out of the first paragraph the sentence:

"Members not residing in North America shall be allocated to District No. 1."

Amend Section 4, by striking out the first paragraph and substituting the following:

"Directors shall be nominated by the Corporate Membership of the geographical districts which they are to represent, and may or may not be resident therein.

"Not later than the fifteenth day of April each year there shall assemble in such geographical districts as are entitled to nominate a Director, and in such zones as are entitled to nominate a Vice-President, representatives chosen by the Local Sections therein, which representatives shall have voting power in proportion to the respective memberships of the Local Sections represented.

"These representatives shall constitute the District or Zone Board and shall nominate a candidate or candidates for the office of Director for the said District or for the office of Vice-President for the said Zone and make announcement thereof to the District or Zone membership.

"If there be but one Local Section in the District said Section may nominate its candidate or candidates for Director in such manner, subject to the approval of the Board of Direction, as it may choose.

"Additional nominations may be made by declaration by at least twenty-five Corporate Members of said District or of said Zone forwarded to the said District or Zone Board within twenty days following said announcement.

"A letter ballot containing the names of the candidates so nominated, upon which the nominees of the District or Zone Board shall be designated, shall be mailed by said Board to each Corporate Member in the District or in the Zone not later than May fifteenth, and the ballots received prior to June tenth shall be canvassed by the said Board, and a report of the result thereof, certified by the said Board, shall be presented by the representatives of the Local Sections of said District or of said Zone to the Annual Conference of Representatives of Local Sections."

Amend Section 4, by striking out the word "President" in the first line of the second paragraph so that the paragraph will read as follows:

"In the event of a tie vote for nominee for Vice-President or Director the names of the persons receiving such tie vote shall be placed on the ticket as 'Official Nominees'."

Add after Section 4 a new Section to read as follows:

"5.—The hereinafter provided Annual Conference of Representatives of Local Sections, at which the representatives shall have voting power in proportion to the respective memberships of the Local Sections represented, shall nominate one or more candidates to fill the office of President to be elected at the next annual election; the written acceptance of each candidate must be obtained prior to his nomination.

"A list of said nominations, together with the list of the nominations for Directors by the several geographical districts, and of the nominations for Vice-Presidents by the several Zones, certified by the Chairman and the Secretary of the said Conference, shall be presented to the Board of Direction not later than the fifteenth day of September.

"The nominations thus made to be known as the 'Official Nominations' shall be such as to provide, with the officers holding over, the officers provided for in Article V."

Renumber Sections 5, 7, 8, 9, and 10, and amend Section 6 to read Section 7, and strike out in the second paragraph the words "showing also thereon the results of the 'second ballot',".

Amend Article VIII.—Meetings:

Add a new section to read as follows:

"5.—There shall be held, during the month of July or August, an Annual Conference of Representatives from the Local Sections to consider the welfare of the Society and its members and to report thereon to the Board of Direction; one representative thereto from each section shall be allowed traveling expenses within Continental United States on a mileage basis.

"The Annual Conference of Representatives of Local Sections shall elect from among its members a Chairman and a Secretary to serve for one year beginning on the first day of the following November. At said Annual Conference a majority of the representatives shall constitute a quorum; if at said Annual Conference a quorum is not present, then such representatives as are present shall call an adjourned meeting."

Amend Article IX.—Local Sections:

Strike out the first paragraph and substitute the following:

"There shall be established in each geographical district of the Society one or more Local Sections. These Sections shall have powers and act under such rules and regulations as the Board of Direction may prescribe.

"Each member of the Society shall identify himself with a Local Section in the district in which he resides, or, in default of voluntary action, shall be assigned to the most suitable section in said district by the Board of Direction."

Strike out of the second paragraph the word "may" in the first line and substitute the word "shall".

These amendments were signed by Richard L. Humphrey, W. L. Stevenson, Howard E. Moses, Christian L. Siebert, G. Douglas Andrews, H. R. Stocker, J. Warren Fortenbaugh, Francis S. Friel, C. A. Emerson, Jr., W. D. Uhler, H. E. Hilts, P. M. Tebbs, J. R. Hoffert, Albert O. True, Samuel T. Wagner, Clark Dillenbeck, Percival S. Baker, W. T. Hopkins, Ralph J. Lawrence, Edwin F. Dawson, Henry C. Smith, Benjamin Franklin, A. R. Lindsey, W. S. Nichols, Joseph C. Wagner, Fred C. Dunlap, G. Roscoe Heap, I. Orlian, J. A. Vogleson, Maurice A. Webster, Stanley H. Wright, Henry H. Quimby, Norman L. Stamm, J. W. Rowland, John J. L. Houston, Stephen Harris, T. Nelson Spencer, William H. Crawford, William Easby, Jr., Edward U. Smith, Elbert S. Tillotson, James W. Follin, Lewis R. Ferguson, and S. C. Hollister.

ITEMS OF INTEREST

The Committee on Publications will be glad to receive communications of general interest to the Society, and will consider them for publication in *Proceedings* in "Items of Interest". This is intended to cover letters or suggestions from our membership concerning matters which are not of a technical character. Such communications, however, must not be controversial or commercial.

THE ENGINEERING FOUNDATION

The Engineering Foundation was established in 1914 "for the furtherance of research in science and engineering, or for the advancement in any other manner of the Profession of Engineering and the good of mankind", and for the following purposes: To promote and support worthy researches related to engineering in all its branches; to establish and operate engineering research laboratories, if funds be provided therefor; to co-operate with National Research Council and the Engineering Societies in the stimulation and co-ordination of scientific research.

Endowment Funds Needed.

The Foundation needs a large increase of endowment. It is obliged frequently to refuse to support research projects brought to it because it lacks funds. Gifts of $1 000 or more are desired. Each donor of $250 000 or more will be honored as a Founder. A gift of $50 000 has been offered contingent on the receipt of nine other gifts of $50 000 each. Gifts to the Foundation are exempt from income tax. A gift for research is a productive investment.

The Foundation is compiling a directory of the hydraulic laboratories of the United States, and is planning an investigation of industrial education and training. It undertakes useful researches which do not promise profits sufficient to tempt industrial organizations to undertake them, researches which should be made under disinterested auspices, and researches which lie outside the province of Government bureaus.

The Engineering Foundation is administered under the auspices of the United Engineering Society, the American Society of Civil Engineers, the American Institute of Mining and Metallurgical Engineers, the American Society of Mechanical Engineers, and the American Institute of Electrical Engineers, by a board of thirteen representatives of these Societies, and three members at large.

A progress report of the Foundation, a form of Deed of Gift, and other information will be sent by the Secretary, Alfred D. Flinn, M. Am. Soc. C. E., 29 West 39th Street, New York City, on request.

Elections to Honorary Membership

At the meeting of the Board of Direction on October 10th, 1921, the Tellers appointed to canvass the Ballots for Honorary Members reported the election of Howard Adams Carson, M. Am. Soc. C. E., Luigi Luiggi, M. Am. Soc. C. E., and Charles Prosper Eugene Schneider as Honorary Members of the Society. The following brief sketches of their lives and work are presented herewith for the information of the membership.

Howard Adams Carson.

Howard Adams Carson, M. Am. Soc. C. E., was born at Westfield, Mass., on November 28th, 1842. He was graduated from the Massachusetts Institute of Technology in 1869 with degree of Bachelor of Science; in 1906 he received the honorary degree of Master of Arts from Harvard University.

In 1871, Mr. Carson acted as Assistant Engineer for the Water-Works Department, Providence, R. I., and, in 1873, had charge of sewer construction for that city. He acted as Principal Superintendent of Construction, in Boston, Mass., for the main drainage system of that city. He was Designer and Chief Engineer for the Charles River Valley sewerage for about twenty cities and towns.

Mr. Carson was Chief Engineer of the Boston Transit Commission from 1894 to 1909. He had charge of the building of the Boston Tremont Street Subway, the first electric car subway constructed in the United States, the East Boston Tunnel, the first submarine tunnel built for such service in the United States, and the Washington Street Tunnel. He has acted as Consulting Engineer for various engineering projects, including the double-track railway tunnel under the Detroit River, at Detroit, Mich.

The following extract from the Fifteenth Annual Report of the Boston Transit Commission, dated June 30th, 1909, is of interest:

"Other qualities than those of technical skill have been shown by him in an equally notable degree. His tact in dealing with public and private interests, which in the progress of these works were of necessity seriously affected, his painstaking fairness in his relations with contractors, his skill in organizing and directing the engineering staff, his executive force, and his unflinching willingness to bear every proper responsibility of his office, have all been conspicuous. Scrupulous honesty and impartiality, moderation and modesty in all things, faithfulness to the point of extreme self-sacrifice have been characteristic of his service. At the termination of that service this Commission records its admiration of his professional skill and its high regard for his personal character."

Mr. Carson is a Trustee of the Massachusetts Institute of Technology, Member of the Institution of Civil Engineers of Great Britain, and Past-President of the Boston Society of Civil Engineers and of the Alumni Association of the Massachusetts Institute of Technology.

Luigi Luiggi

Luigi Luiggi, M. Am. Soc. C. E., was born at Genoa, Italy, in 1856. He was graduated from the University of Genoa, Italy, in 1875, with the degree of

Doctor of Physics and Mathematics, and from the Royal College for Civil Engineers with the degree of Civil Engineer, in 1878. The following year he became a Cadet, Italian Royal Artillery, Coast Defenses, and in 1880, was appointed Member of the Royal Corps of Civil Engineers of Italy. In this capacity he was sent to England for two years for special practice in maritime engineering, lighthouses, etc.

In 1882, Mr. Luiggi was appointed Resident Engineer of Harbor Works at Genoa, Italy, in charge of breakwaters, quay walls, docks, etc. In 1887 he was made Director of Works for the design and construction of the two dry docks at Genoa, which were built under compressed air. Upon completion of these docks, he was promoted to the position of Engineer-in-Chief, Royal Corps of Civil Engineers, and appointed Private Technical Adviser to the Minister of Public Works at Rome.

Mr. Luiggi was a delegate of the Public Works Department of the Italian Government to the Engineering Congress at Chicago, Ill., in 1893. The following year he acted as Engineer-in-Chief at Leghorn, Italy, for all maritime works in the Tuscan and Roman Provinces. In 1896 he was called by the Argentine Government to protect the Military Port at Bahia Blanca, Argentina, and all the maritime works and lighthouses along the coast of Patagonia to the Strait of Magellan, and in 1889 was appointed Director General of all these works. These were completed in 1905, and after being called by the Uruguayan Government to inspect and advise about the best methods to expedite works for the Port of Montevideo, he returned to Italy to be appointed by the Italian Government as a member of the Consulting Board for Public Works and of the Special Board for the Managament of the Italian State Railway.

Mr. Luiggi was successively Chief Engineer, Acting Inspector General, and Inspector General of the Royal Corps of Italian Civil Engineers. He is Professor of Hydraulic Engineering at the University of Rome, and a member of the International Technical Committee for the Suez Canal. He was appointed Engineer Delegate on the Industrial Commission to the United States, and contributed to the International Engineering Congress at San Francisco, Cal.

Mr. Luiggi served during the World War as Colonel, Artillery Corps, Italian Army, and as a member of the Committee for Munition Works, receiving a gold medal from the Minister of Munitions for distinguished service.

Charles Prosper Eugene Schneider

Charles Prosper Eugene Schneider was born at Le Creusot, France, on October 29th, 1868. He is a grandson of the founder of France's great machine shops at that place.

Mr. Schneider is managing owner of the Creusot Works, and developed the manufacture of the celebrated soixante-quinze (75-mm.) rapid-fire gun, and also howitzers, railway mounted guns and other ordnance used in the World War.

On November 24th, 1919, Mr. Schneider was honored by the presentation of a gold medal by the Mining and Metallurgical Society of America for "his distinguished work in metallurgy of iron and steel, and especially for his development of the 75-mm. gun, to which a large part of the success of the

French and of our own men in France during the present war, is attributed." The Gold Medal Committee of that Society, in its report of Mr. Schneider's career, made the following statements:

"Whilst giving to peace industries all the attention necessitated by the progress in science and the continual improvement in industrial methods and products, Eugene Schneider, with a perspicacity that recent events have justified in a striking manner, especially directed his efforts towards the creation in France of a war industry able to counterbalance, when the time came, the enormous power which the German war industry had established * * * Already in 1895, the Schneider establishments made special efforts to realize and improve heavy and light field ordnance, known as quick-firing, the appearance of which called forth a revolution in the armament and tactics of modern artillery.

"The war materials delivered in very large quantities during the war to the French and Allied Governments, are of the most varied types: Field guns and howitzers (heavy and light types), siege guns, large caliber guns on railway mounts, tanks, shells, cases, fuses, explosives, torpedoes, sights, submarine and airplane engines, armor plate, etc.

"Besides the technical and industrial development of the works, Eugene Schneider has given his attention to social economics as begun by his ancestors for the welfare of their employees; all questions pertaining to the interest of workmen have been the subject of his constant attention and, very often, received solutions which were very much in anticipation of recent laws."

Mr. Schneider visited the United States during the latter part of 1919 as Chairman of the French Economic Mission in the interest of closer co-operation between France and this country.

Mr. Schneider is Past-President of the Iron and Steel Institute of Great Britain, and Honorary President of the Comité des Forges, France.

Conference on Elimination of Excess Variety and Standardization of Vitrified Paving Brick

The Conference of users and makers of vitrified paving brick was called at the suggestion of the National Paving Brick Manufacturers Association, representatives of which met with representatives of the Department of Commerce and with representatives of the U. S. Chamber of Commerce in a preliminary conference to determine the areas of standardization possible in this particular industry. As a result of this preliminary meeting the manufacturers, under the general direction of the Department of Commerce, instituted a variety survey of the vitrified paving brick industry. This survey formed the basis for the meeting held November 15th, 1921, called by the Department of Commerce.

A permanent committee to be known as the Committee on Simplification of Variety and Standards for Vitrified Paving Brick of the Department of Commerce was created for the purpose of making such other eliminations as shall be mutually acceptable to producer and consumer.

In response to an invitation delegates were present, representing manufacturers, architects, engineers, and the Government departments, W. D. Uhler, M. Am. Soc. C. E., Chief Engineer of the Pennsylvania State Highway Department, representing the Society.

In opening the Conference, F. M. Feiker, Assistant to the Secretary, United States Department of Commerce, read a copy of the invitation and in concluding his remarks stated that the Paving Brick Manufacturers Association had made a complete and exhaustive study of the sizes and varieties of paving brick, and that the purpose of the Conference was to bring together the representatives of the manufacturers, engineers, and all those who had any specifying or buying relations to this problem, and to discuss it from the point of view of eliminating excess sizes and varieties.

Secretary Hoover, addressing the conferees, stated in substance that the proposal under consideration was no new idea in American industry, but that it comes up in its best form on this occasion because it is inspired by the manufacturers themselves.

He stated further that engineers have been united in the feeling that there is a great area of waste in American industry that can only find correction at the hands of the manufacturers by a purely voluntary action on their part.

He also stated that there are a number of manufacturers carrying on their own surveys, who are in consultation with the Department; but that to make any of this effective does not lie entirely with the manufacturers, who must have the co-operation of outside groups. Continuing, he stated that this is the first time that the Department of Commerce has attempted to bring the groups together, first, the manufacturer, then those who dominate his consumption, so that results can be obtained.

The Conference through a process of elimination, using as its basis a maximum size of brick, 4 in. by $3\frac{1}{2}$ in. by $8\frac{1}{2}$ in., and as a minimum, 3 in. by 3 in. by $8\frac{1}{2}$ in., reduced the varieties for consideration from 66 to 20.

Considerable debate ensued concerning further elimination, and it was decided to appoint a committee for the purpose of considering the remaining 20 varieties.

In approaching the subject the Committee considered it desirable, if possible, to reduce the number of sizes so that all brick could be cut out of two clay columns, one 3 in. and the other 4 in. high.

It felt that the present demands are such that there must be placed at the disposal of engineers brick to make a wearing surface either 3, $3\frac{1}{2}$ or 4 in. in depth. In the smaller cities, a 3-in. pavement is wanted. The larger cities require a 4-in. brick. The State highway departments find 3 in. too shallow for their traffic and 4 in. deeper than necessary, and are, therefore, adopting a $3\frac{1}{2}$-in. depth.

With these three depths considered as imperative, the Committee deemed it desirable to eliminate only 9 of the varieties over and above those eliminated previously.

With these eliminations the number of standard varieties would be 11, and the number of sizes 4, as follows:

Width, in inches.	Depth, in inches.	Length, in inches.
$3\frac{1}{2}$	4	$8\frac{1}{2}$
3	4	$8\frac{1}{2}$
$3\frac{1}{2}$	$3\frac{1}{2}$	$8\frac{1}{2}$
$3\frac{1}{2}$	3	$8\frac{1}{2}$

The varieties, therefore, which would be retained are as follows:

Variety.	Width, in inches.	Depth, in inches.	Length, in inches.
Plain wire-cut brick (vertical fiber lugless)	3	4	8½
	3½	4	8½
Repressed lug brick	3½	3½	8½
	3½	4	8½
Vertical fiber lug brick	3	4	8½
	3½	4	8½
Wire-cut lug brick (Dunn)	3½	3	8½
	3½	3½	8½
	3½	4	8½
Hillside lug brick (Dunn)	3½	4	8½
Hillside lug brick (repressed)	3½	4	8½

The Committee believes that further reduction of varieties is desirable, but that this should follow after further study and after the idea of standardization in vitrified paving-brick sizes has become well impressed on the paving field. To go too far at the start is likely to arouse such strenuous opposition as to defer for a protracted time the good results which are desired from this Conference.

Following the adoption of the Committee's report, the question as to the usual variations incident to the manufacture of brick was discussed at length. In order to avoid the necessity of being held to exact dimensions, the following resolution was adopted:

"The sizes stated in this report are to be regarded as nominal and subject to the usual variation of ⅛ in. in width and depth, and ¼ in. in length."

Speaking for the Department, Mr. Feiker impressed on the Conference the necessity of invoking a follow-up system to insure the adoption of the standardization embodied in the resolutions of the Conference; to effect a greater degree of contact and co-operation between the Department of Commerce and the various organizations and manufacturers incident to the paving brick industry; and to consider further eliminations in the existing varieties of brick.

This proposal was accepted by the Conference and in conformity therewith a Committee of ten was appointed for that purpose. The personnel of this committee will be composed of a representative of the following organizations: American Society of Civil Engineers; American Association of State Highway Officials; American Society of Municipal Improvements; American Society for Testing Materials; Federated American Engineering Society; National Paving Brick Manufacturers Association; U. S. Chamber of Commerce; U. S. Bureau of Public Roads; U. S. Bureau of Standards; and U. S. Department of Commerce.

Industrial Standardization in Germany

Insufficient attention has been given to the rôle which standardization is playing in German industrial reconstruction. The German industries are planning and are carrying out a far-reaching program of standardization as a necessary step in building up an unprecedented industrial structure which

must rest in large measure on an extensive foreign trade. In no other country except Great Britain is standardization work being done on such a scale, or with an intensity, comparable to that in Germany.

The German work is of special interest to those responsible for the management of American industries, not only because of its importance, but also because of the similarity in the historical conditions surrounding the National standardization movements in Germany and in America.

Prior to 1917 a vast amount of standardization work had been carried out in Germany by individual companies, and by engineering societies and industrial associations, but, as was the case in America before the organization of the American Engineering Standards Committee, the work had not been unified along National lines.

As has been the case with all the other National standardizing bodies except the British, which was organized in 1901, the success of the standardization work carried out by the various countries during the World War as a part of their National conservation programs, was a chief cause of the formation of the Central German Body. It is called the "Normenausschuss der Deutschen Industrie," and was organized by the "Verein Deutscher Ingenieure," in 1917 at the suggestion of the German Government. The present membership consists of engineering societies, industrial associations, and Government ministries, and, in addition, there are 700 firms which are contributing members. The work of the Normenausschuss deals only with those subjects which concern two or more industries or branches of industry.

The standards are issued under the general designation of German Industrial Standards. The Germans were the first of the National bodies to publish standards in loose-leaf form. The work is so divided as to make each sheet as nearly independent as possible. Firms purchase these sheets in quantity, issuing them directly to designers, draftsmen, and foremen for use as working drawings and data sheets.

From almost the first the Normenausschuss has had a periodical publication dealing with standardization. Formerly, it was a separate publication, but now it forms a section of *Der Betrieb,* a journal dealing with the general question of production and efficiency engineering. The communications from the Normenausschuss (*Mitteilungen*) form a separate section in this journal which appears semi-monthly, and has a circulation of 8 000 copies.

The organization provides an extensive information service on standardization work in Germany and other countries, which is available to the industries as well as to their working committees.

Organization and Methods of Work.—There is a Main Committee composed of representatives of the various National organizations supporting the movement, and a smaller Executive Committee. The detailed technical work of each project is in the hands of a working committee which in Anglo-Saxon countries would be called a "sectional committee," that is, a committee made up of representatives of all bodies interested in the particular subject in hand.

Proposals for new subjects for standardization must come from some responsible body. The industry concerned is consulted, generally by a conference of the various organizations interested, to determine whether it is

the consensus of opinion that the work should go forward. In case it is decided to undertake the work, the conference designates the chairman of the working committee.

The central office digests the information available on the subject for the use of the working committee. When agreement is reached in the committee on the draft of a standard it goes to the central office for editorial work. There it is scrutinized to see whether it is consistent with other standards; whether points have been included which concern other working committees; whether the drawings and nomenclature are in approved form, etc. After this editorial checking, the central office has the draft of the standard put into proof form. It is then reviewed by an official clearing house committee, which contains a representative from each major line of work being carried on by working committees. If any change in substance has been made, it goes back to the working committee. If no such change has been made, it is published in the *Mitteilungen,* the official publication of the Normenausschuss, as a tentative proposal. On recommendation of the working committee, the standard is mailed to the members of the Executive Committee with a supporting statement. With their approval it is then republished as an official proposal. Six weeks are allowed for criticism when a standard is finally published unless additional important criticisms are received.

Work in Special Industries.—The foregoing refers to the work of the central body only, which is limited to subjects common to two or more industries. In addition there are about fifteen organizations known as special industry committees, each of which deals with the standardization work peculiar to a single industry, such as shipbuilding, electrical, agricultural, automative, elevator, locomotive, paper, textile, and wood-working.

These committees are closely affiliated with, but not strictly an organic part of, the central body. They are organized not by the Normenausschuss, but by one or more technical or trade associations concerned with the particular subject in hand. Standards formulated by the special industry committees are published by the organization responsible. In most cases the final standards are published in loose-leaf form modeled closely after that of the standards issued by the Normenausschuss itself. These standards are submitted to the Normenausschuss before publication, in order to keep them consistent with the regular series of German industrial standards.

The volume of work being carried out through these special industry committees appears to be at least as great as that under the direct control of the central body.

Characteristics of Continental Work.—Looked at broadly, and with exceptions such as must always be made in general statements of the kind, the Continental countries are going much further into dimensional standardization than has been done in Anglo-Saxon countries. This includes interchangeability of supplies and of machine elements, the interworking of parts and of related apparatus made by different makers, and the interchangeability, so far as the use is concerned, of complete machines and apparatus. By far the greater part of the work of the Normenausschuss is dimensional, great attention being paid to such matters as machine elements, screw-threads, bolts

and nuts, standard diameters, and systems of limit gauging. Some of the special industry committees are active in the elimination of types, sizes, and grades of manufactured products.

The Germans have not yet had their standards translated into foreign languages for use in export, as the British are doing, but they are now giving consideration to this question.

As typical examples of the German work, their system of "preferred numbers" and their standard series of handles may be mentioned. The first is a simple system of numbers for use in all new standardization work, in which gradated numerical values are required, such as standard gradated diameters of pulleys, thicknesses of plates, or capacities of machines. It is believed that its use will lead to great economies in material, in reducing the number of sizes, ranges, etc., simplify the carrying of stocks and facilitate interchangeability. It may be shown theoretically that, under average conditions, a given number of standard sizes laid out according to these numbers, will be better fitted to any series of jobs taken at random, than would be the same number of sizes laid out in any other way, and this with a minimum of material.

The standard handles are of two shapes, each adapted to a particular method of use, and there is a series of sizes for each shape. The profiles have been worked out with the extreme care, an efficiency engineer having been employed to make time-motion studies to determine the exact profile that would insure the greatest accuracy in operation with the minimum fatigue of the workman's hand.

ACTIVITIES OF LOCAL SECTIONS*

Regular Meeting of the San Francisco Section

The Ninety-ninth Regular Meeting of the San Francisco Section was held at the Engineers' Club on October 18th, 1921; President F. R. Muhs in the chair: H. D. Dewell, Secretary; and present, also, 80 members and guests.

Mr. Ned D. Baker, Chairman of the Excursion Committee, reported that his Committee was planning an excursion to Pittsburg, Cal., to witness the tests of concrete roads now nearing completion at that place.

The Secretary presented a letter from Mr. J. W. Mahoney, Secretary-Treasurer of the San Francisco Electrical Development League, announcing the project for the construction of an Engineering and Industry Building, and asking for the financial co-operation of the Section. On motion, duly seconded, it was decided to refer the matter to the Board of Directors for consideration and decision, with authority to appoint, as requested, one member to each of the three committees for the work, and to contribute to the project a sum not to exceed $100.

President Muhs announced the death of Ralph E. Robson, Assoc. M. Am. Soc. C. E., a member of the Section, on October 14th, 1921.

Mr. H. J. Brunnier, who recently attended an International Convention of the Rotary Clubs in Edinburgh, Scotland, gave an informal talk on his experiences in England, France, Belgium, Germany, Switzerland, and Italy.

The speaker of the evening was Mr. W. H. Kirkbride, Engineer of Maintenance of Way and Structures, Southern Pacific Company, Pacific System, whose subject was "Railroad Tunnels and Their Maintenance". In connection with his address, Mr. Kirkbride exhibited lantern slides showing all stages of tunnel construction, enlargement, concreting, and maintenance.

Regular Meeting of Colorado Section

The 120th Regular Meeting of the Colorado Section was held at the Metropole Hotel, Denver, Colo., on November 14th, 1921; President A. N. Miller in the chair; Walter L. Drager, Secretary; and present, also, 10 members and 7 guests.

The minutes of the 119th regular meeting were read and approved.

The Secretary presented a copy of H. R. 7541 which provides for a commissioned status to sanitary engineers in the Public Health Service of the United States, and read letters from various Local Sections urging the endorsement of this bill by the Colorado Section. On motion, duly seconded, the bill was endorsed by the Section and the Secretary was instructed to notify the Congressmen from Colorado of this action.

Mr. H. L. Thackwell presented a plan for the construction of an Engineers' Building in Denver. After discussion by those present, it was decided, on motion, duly seconded, to appoint a committee to assist Mr. Thackwell in securing additional data on the matter for presentation to the members of the Section.

* For list of Local Sections, Officers, etc., see p. 966.

On motion, duly seconded, it was decided to postpone the discussion of the Progress Report of the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete published in the August, 1921, *Proceedings* of the Society.

The address of the evening was on "The Development and Application of the Ball Bearing" by Mr. R. H. Fox, who illustrated his lecture with lantern slides. The subject was discussed generally by the members and guests present.

Meetings of Cleveland Section

A meeting of the Cleveland Section was called to order on October 12th, 1921, at the Winton Hotel; Past-President W. P. Brown in the chair; George H. Tinker, Secretary; and present, also, 9 members.

The minutes of the meeting of September 14th, 1921, were read and approved, but no other business was transacted.

MEETING OF NOVEMBER 9TH, 1921.

The regular meeting of the Cleveland Section was held at the Winton Hotel on November 9th, 1921; F. C. Osborn in the chair; George H. Tinker, Secretary; and present, also, 8 members.

The minutes of the meeting of October 12th, 1921, were read and approved.

The Secretary presented communications from the Los Angeles Section and the Service Engineers' Committee of the U. S. Public Health Service relative to H. R. 7541 as to the status of civilian engineers in the U. S. Public Health Service. On motion, duly seconded, both communications were referred to the Legislative Committee.

On motion, duly seconded, the Secretary was instructed to write to Mayor-elect Kohler urging the appointment of an engineer as Director of Public Service. It was also voted that the Cleveland Engineering Society be urged to take similar action.

Mr. B. R. Leffler reported that the Special Committee of the Society on Bridge Design and Construction would shortly present a report, and urged the members of the Section to submit written discussions thereon for publication by the Society.

Annual Meeting of Iowa Section

The Third Annual Meeting of the Iowa Section was called to order at Des Moines, Iowa, on November 17th, 1921, at 11 A. M.; President C. S. Nichols in the chair; R. W. Crum, Secretary; and present, also, 15 members and 3 guests.

On motion, duly seconded, the bill (H. R. 7541) now before Congress providing a commissioned status to sanitary engineers in the Public Health Service of the United States was endorsed, and the Secretary was instructed to bring this endorsement to the attention of the U. S. Senators and Representatives from Iowa.

The following officers for 1922 were elected: President, J. H. Dunlap; Vice-President, J. S. Morrison; and Director, O. W. Crowley.

The afternoon was spent in an inspection trip to the plant of the Pyramid Portland Cement Company now under construction.

The meeting was reconvened at 4 P. M., and was devoted to a general discussion of the activities of the Section.

On motion, duly seconded, the dues of the members for 1922 were fixed at $2.50.

A paper entitled "City Zoning and Its Effect on City Building", by R. E. Edgecomb, Assoc. M. Am. Soc. C. E., Chief Engineer of the Building Department of Omaha, Nebr., was presented by the author.

The final session of the meeting was the Annual Dinner which was held at 6.30 P. M., at the Harris-Emery Tea Room. Following the dinner, Prof. Dunlap gave a brief description of the activities of the Federated American Engineering Societies.

Meeting of Los Angeles Section

A meeting of the Los Angeles Section was called to order at 7.40 P. M., on November 9th, 1921, at the Wilshire Country Club; President H. W. Dennis in the chair; F. G. Dessery, Secretary; and present, also, 57 members and 47 guests.

After introducing some of the guests, President Dennis presented the speaker of the evening, Mr. J. B. Lippincott, who addressed the meeting on "Colorado River Problems with Reference to Flood Control, Silt Control, Power and Irrigation", illustrating his remarks with diagrams and lantern slides. The subject was discussed by Messrs. Hill, Jubb, Dennis, Cronholm, Anderson, Barnard, Miller, Mulholland, Griffin, Thomas, Wheeler, and Sparks. At the conclusion of the discussion, President Dennis, on behalf of the Section, thanked Mr. Lippincott for his instructive address.

On motion, duly seconded, the report of the Committee on Building Laws and Regulations was adopted with some dissenting votes. After discussion of the subject by Messrs. Noice, Flaherty, Wheeler, and Barnard, on motion, duly seconded, this action was reconsidered.

Mr. Blaine Noice, on motion, duly seconded, was requested to submit a Minority Report, at which time the whole subject would again be taken up for consideration.

Mr. G. G. Anderson called attention to the fact that the December meeting was the Annual Meeting of the Section, and suggested that the report of the Committee on Building Laws and Regulations be considered as the program for that meeting.

The Secretary requested the Standing Committees of the Section to submit written reports to be presented at the Annual Meeting.

Regular Meeting of Louisiana Section

The regular meeting of the Louisiana Section was held on October 24th, 1921, at the residence of President Ole K. Olsen in New Orleans; President Olsen in the chair; F. A. Muth, Secretary; and present, also, 13 members and 3 guests.

Relative to the question of licensing engineers, President Olsen presented correspondence from Richard L. Humphrey, M. Am. Soc. C. E., Chairman of the Committee on Licensing Engineers, of the Board of Direction of the Society, together with his reply, in which he stated that the licensing of engineers in Louisiana was already a fact and was operating satisfactorily.

President Olsen also presented correspondence between the Secretary of the Society and himself relative to a newspaper clipping from a Memphis paper referring to flood damage at Jackson, Miss. Concerning this matter, President Olsen stated that he had made a personal examination at the site in Jackson and had reported on it to the Society. On motion, duly seconded, the correspondence was ordered filed.

On motion, duly seconded, the Secretary was instructed to refer the matter of the organization of Student Chapters to Professor W. B. Gregory at Tulane University.

A communication was presented by the Secretary relative to the use of standard letter-heads for local Sections, and on motion, duly seconded, the matter was referred to the President and Secretary with power to act.

The attention of the members was called to the proposed organization of an Engineers' Club in New Orleans, and after discussion by those present, on motion, duly seconded, the Secretary was instructed to send out postal cards to the membership of the Section for expressions of individual opinion on the subject of the tentative organization of an Engineers' Luncheon Club in New Orleans.

Various other matters of interest to the Section were discussed, and after the meeting was adjourned, the members present were entertained at a Smoker by President Olsen.

New York Section Participates in Joint Meeting on the Financing of Engineering Projects

The program for the 1921-1922 season planned by the New York Section was inaugurated on October 19th, 1921, when for the first time in their history the four Founder Societies, through the co-operation of the Local Sections of the Metropolitan District, united in a joint meeting, and considered the subject "Financing of Large Engineering Projects, Including Public Utilities, Industrial, and General Engineering Projects."

The subject was introduced by Messrs. Arthur B. Leach, of A. B. Leach and Company, of New York City, and Philip Cabot, of White, Weld, and Company, of Boston, Mass. In the course of his discussion, Mr. Leach presented figures to show the remarkable change in financial and world conditions since 1914, and Mr. Cabot stated that the engineer, because of his training, tends to develop qualities that make him a failure as a financial manager. The discussion was led by Mr. J. H. Williams, of Day and Zimmerman, Consulting Engineers, and among those who took part in it were Messrs. Calvert Townley, Vice-President of the Westinghouse Company, Lewis H. Nash, A. Korminsky, and Blaney Stevens.

The meeting was held in the Auditorium of the Engineering Societies Building, with Farley Osgood, Chairman of the New York Section of the

American Institute of Electrical Engineers, in the chair, and there were approximately 1 250 members and guests present.

JOINT MEETING OF ST. LAWRENCE SHIP CANAL AND POWER PROJECT

The second joint meeting of the Metropolitan Sections of the Founder Societies was held in the Auditorium of the Engineering Societies Building, on November 14th, 1921.

Addresses on the "St. Lawrence Ship Canal and Power Project" were made by Julius H. Barnes, President of the U. S. Grain Corporation; H. I. Harriman, Chairman of the Massachusetts State Commission of Foreign and Domestic Commerce, Boston, Mass.; Governor Henry J. Allen, of Kansas; Ex-Governor Harding, of Iowa; and Dr. R. S. McElwee, Director of the School of Foreign Commerce, Georgetown University, Washington, D. C. The subject was also discussed by Capt. Charles Campbell.

Organization Meeting of Northeastern Section

At a meeting held on November 12th, 1921, at the Boston City Club, the organization of the Northeastern Section was completed.

On motion, duly seconded, the Constitution, as approved by the Board of Direction, was adopted, together with the By-laws.

The following officers were elected: Chairman, Frank B. Sanborn; Vice-Chairman, Walter C. Voss; Secretary-Treasurer, Charles W. Banks; and Members of the Executive Committee, Leonard C. Wason and James H. Manning.

Meeting of Portland Section

The meeting of the Portland Section was called to order on October 28th, 1921, at the University Club; President M. E. Reed in the chair; C. P. Keyser, Secretary; and present, also, 33 members and 5 guests.

The minutes of the meeting of September 16th, 1921, were read and approved.

President Reed called attention to the matter of the report on the licensing of engineers for Director Richard L. Humphrey's committee, and called on Mr. F. S. Baillie, a member of the State Board for Registering Engineers, for a statement, which he made. On motion, duly seconded, the Secretary was instructed to report to Director Humphrey that in the opinion of the members of the Portland Section the Oregon law for licensing engineers has been in effect too short a time for a definite conclusion to have been reached relative to its beneficial and detrimental effects.

A general invitation was extended to members of the Section to be the guests of the Portland Chapter of the Associated General Contractors at a dinner at the Multnomah Hotel on November 2d, 1921. President Reed on behalf of the Section accepted the invitation and appointed Past-President Newell to represent the Section at the banquet.

A paper entitled "The Treatment of the Cascades Slide" was presented by Mr. Samuel Murray, and the subject was discussed by Messrs. J. P. Newell, W. G. Brown, D. D. Clarke, and Ira A. Williams.

Relative to the selection of a site for the 1925 Fair, Mr. J. A. Currey of the Committee advised that no progress had been made relative to the matter since the questionnaire was filed with the Fair Committee on Sites. Mr. Currey also commented briefly on the working of the Building Code and recommended some movement toward relief for the Building Inspector's Office from an unwarranted burden imposed by filers of deficient or irregular plans.

Fall Meeting of Texas Section

The Fall Meeting of the Texas Section was held on October 28th, 29th and 30th, 1921, at the Hotel Jefferson, Dallas, Tex., and was attended by 98 members and guests.

The Business Meeting was held on Friday morning, October 28th, at which an address of welcome was made by the Mayor of Dallas. The presentation of the Annual Address by President J. H. Brillhart was followed by the business session, at which officers for the ensuing year were elected as follows: President, E. B. Cushing; First Vice-President, E. E. Sands; Second Vice-President, W. J. Powell; and Secretary-Treasurer, E. N. Noyes. The remainder of the morning was devoted to the reading of papers and discussions.

The afternoon was spent in an inspection trip to the plant of the Trinity Portland Cement Company, at which a barbecue luncheon was served by the Company, after which the members and guests were taken for an automobile ride over the new concrete highway in Dallas and Tarrant Counties.

In the evening the party was tendered a dinner by the Technical Club of Dallas on the Jefferson Roof Garden. The dinner was followed by the presentation of illustrated papers, among which was one on "The San Antonio Flood" by Mr. Terrell Bartlett. The presentation of the papers was followed by music and dancing.

The morning of Saturday, October 29th, was taken up with the presentation and discussion of papers on various technical subjects. Luncheon was served on the Jefferson Roof Garden, and was followed by an address on "Constitutional Handicaps of Texas Cities" by Mr. Tom Finty, Jr. Through the courtesy of Mr. Robert Brown, the members of the party were then taken through the Magnolia Building, after which the afternoon and evening were devoted to automobile trips to various points of interest around the city.

On Sunday, October 30th, the visitors were also entertained by automobile excursions to various places.

The papers presented at the meetings included the following: "Present Condition and Operation of Sewage Disposal Plants in a Number of Texas Cities" by M. C. Erwin, who illustrated his remarks with lantern slides; "Friction Head in a Tuberculated 12-Inch Cast-Iron Conduit" by Messrs. M. C. Welborn and John B. Hawley; "Irrigation and the Wichita Falls Project" by Vernon L. Sullivan; "City Planning" by E. A. Wood; "Makeshift Water Treatment Plant at Breckenridge" by John B. Hawley; "The Rôle of the Sanitary Engineer" by Louva G. Lenert; "Asphalt Macadam Pavements in Mineral Wells" by W. W. McLendon; "Water Supply in East Texas" by H. N. Roberts; and "Types of Cracks in Bituminous Pavements and Their Causes" by W. P. Bentley.

EMPLOYMENT SERVICE OF THE FEDERATED AMERICAN ENGINEERING SOCIETIES

An Engineering Societies Service Bureau was established December 1st, 1918, as an activity of Engineering Council, managed by a board made up of the Secretaries of the four Founder Societies, funds for its maintenance being provided by these Societies. On January 1st, 1921, this Bureau was taken over by The Federated American Engineering Societies and is now known as the Employment Service of that organization. It is co-operating with engineering organizations in all parts of the country and is desirous of increasing such co-operation by working with local engineering associations and clubs. Members of the American Society of Civil Engineers who desire to register should apply for further information, registration forms, etc., to Walter V. Brown, Manager, Engineering Societies Building, 29 West 39th Street, New York City. In order to be included in the list published in *Proceedings,* copy must be received on or before the first Wednesday of each month. All communications should be addressed to Mr. Brown.

EMPLOYMENT BULLETIN

POSITIONS AVAILABLE

ASSISTANT PROFESSOR OF HYDRAULICS. Must have had experience in hydraulic laboratory investigations and be competent to take charge of laboratory. Location, Northwest. X-1320.

CIVIL ENGINEER REPRESENTATIVE, cities over 25 000; part time; commission basis; must be high caliber; consulting building engineer preferred. X-1380.

MEN AVAILABLE

DREDGING SUPERINTENDENT. Eighteen years' experience in design and operation of all types of dredging equipment. Has had successful charge of many large projects. Desires U. S. or foreign engagement. CE-277.

GRADUATE ENGINEER, Assoc. M. Am. Soc. C. E., age 34, married. Ten years' wide experience, construction, estimating, designing, valuation of buildings and general construction. Desires responsible position. Executive or estimating position preferred. CE-278.

CIVIL ENGINEER. Fifteen years' experience on municipal and highway work, some building, would like change. Good draftsman and concrete designer. CE-279.

CIVIL ENGINEER, Assoc. M. Am. Soc. C. E., technical graduate, age 35. Ten years' experience on responsible work as follows: Hydraulic and power work, location, design, and construction; highway work, location, construction, and maintenance; railway work, location, construction, and valuation. Desires position on the teaching staff of university. CE-280.

CIVIL ENGINEER, M. Am. Soc. C. E., technical graduate, age 35. Twelve years' broad experience on responsible work as follows: Industrial plants, steel and reinforced concrete buildings, subways, viaducts, water supply, sewerage systems, power plants, railways, piers, and warehouses. Has had charge of designs, estimates, and construction work. Capable organizer and executive. CE-281.

GRADUATE CIVIL ENGINEER AND CONSTRUCTION SUPERINTENDENT, Assoc. M. Am. Soc. C. E., age 34, degree 1908. Twelve years' experience, roads, bridges, surveys, sewers, water-works, and concrete industrial buildings. Experience includes design, inspection, and superintendence. Two years in charge of war work for Construction Division, U. S. A. Available at once. Location immaterial. CE-282.

CIVIL ENGINEER, M. Am. Soc. C. E., college graduate. Twenty years' broad practical engineering and contracting experience on water-works, sewers, highways, hydraulics, and general engineering, with engineers, contractors, and utility holding companies; investigations, reports, design, construction, appraisals. Excellent record and references. Will consider any proposition, engineering, or associated work. Member, American and New England Water Works Associations, etc. CE-283.

JUNIOR, Am. Soc. C. E., age 25, married. Technical graduate with good grasp of mathematics and mechanics wants position offering chance of permanence and advancement as Draftsman for consulting engineer or as Assistant Engineer on construction work. Experience: three years' land and mine surveying; drafting, and building construction (steel and reinforced concrete); also, formerly, Instructor in

Sheffield Scientific School, Yale University. Glad to have reference asked of any employer. Hard worker, reliable. Formerly, Lieutenant, Engineers, A. E. F. Now employed, but available on short notice after January 1st, 1922. CE–284.

HYDRAULIC ENGINEER AND WATER-WORKS MANAGER. Twenty years' experience in the design, construction, and operation of water-works and allied structures. Has directed inventories and appraisals and investigations for new projects. Available January 1st, 1922. Location immaterial. Can furnish many excellent references. CE–285.

GRADUATE CIVIL ENGINEER, 1910, M. Am. Soc. C. E., and A. I. M. E., age 33, married, desires executive position. Experience: General engineering, administrative, commercial, foreign developments and negotiations. Has traveled throughout world, principally Far East and South America. Would consider investing in business. Open for all or part time. Eastern interviews. Salary and references in conference. CE–286.

ENGINEER AND EXECUTIVE, M. Am. Soc. C. E., age 32, married. Twelve years of well varied experience on design and construction, with special training on projects connected with the paper industry. Desires executive position with manufacturer or contractor in the East. CE–287.

ENGINEER, Assoc. M. Am. Soc. C. E., age 29, married. Experience covers railroad construction, highway design and construction, railroad traffic in connection with supplying construction materials, selection and handling and storage of construction materials, problems relating to management of construction. Location preferred is Middle West. Date available, about December 15th. CE–288.

CONSTRUCTION ENGINEER OR SUPERINTENDENT, Assoc. M. Am. Soc. C. E., age 33, married. Thoroughly competent of supervising any class of construction; efficient executive and organizer; broad experience as general superintendent, construction superintendent, and resident engineer on monumental and industrial buildings, steam, and hydro-electric power stations, dams, hydraulic work, difficult foundations, concrete work, structural steel, and the installation of mechanical and electrical equipment. Available at once; location immaterial. CE–289.

ANNOUNCEMENT

The Reading Room of the Society is open from 9 A. M. to 6 P. M., and from 7 P. M. to 10 P. M., every day, except Sundays, New Year's Day, Washington's Birthday, Memorial Day, Fourth of July, Labor Day, Thanksgiving Day and Christmas Day; during July and August, it is closed at 6 P. M.

FUTURE MEETINGS

January 4th, 1922.—8 P. M.—The regular business meeting of the Society, together with a Conference on the "National Housing Problem", of one or more sessions, will be held at the Engineering Societies Building.

ANNUAL MEETING

The Sixty-ninth Annual Meeting will be held at the Headquarters of the Society, 33 West 39th Street, New York City, on Wednesday, Thursday, and Friday, January 18th, 19th, and 20th, 1922.

The general arrangements for the Annual Meeting are in the hands of the following Committees:

Committee of the Board of Direction

Richard L. Humphrey, *Chairman,*

John P. Hogan
Francis Lee Stuart
A. M. Hunt
Elbert M. Chandler

Local Committee

J. P. H. Perry, *Chairman,*

Laurence A. Ball
Robert Ridgway
Charles Hansel
Merritt H. Smith
Nelson P. Lewis
D. L. Turner

SECOND MEETINGS OF THE MONTH

Under authority given by the Board of Direction at its meeting of August 9th, 1920, the Acting Secretary has made an arrangement with the New York Section whereby the latter will take over the second meeting of the month, and will thus hold its own meetings on the third Wednesday of each month, except January and May, when they are held on the second Wednesday.

The programmes of the New York Section* are similar to those heretofore offered by the Society's Committee on Second Meeting of the Month, and it is understood that all members of the Society are invited to attend the meetings regardless of whether or not they may be members of the Section. This arrangement gives each member the same privilege of attendance at meetings which he has heretofore enjoyed, and is deemed especially desirable since there has been considerable doubt as to the attendance that might develop at the several meetings if three were held in each month.

* *Proceedings,* Am. Soc. C. E., October, 1921, p. 803.

"TRANSACTIONS" FOR SALE

It is possible to secure a fairly complete set of the *Transactions* of the Society for a very reasonable price as, owing to limited storage space, the Board of Direction has decided to dispose as rapidly as possible of surplus stock.

Some volumes are entirely out of print. Of those available, the following can now be furnished to *members of the Society* for the prices noted:

Vols. 2, 6, 9-10, 15-20, 22, 24-27, 29-42, 44..................	(30 Vols.)	$50
" 45, 49-53, Parts A-F of 54, 55-67, 69-70, 72-79............	(35 ")	$50

It is suggested that members wishing these volumes send in their orders promptly, as the supply of certain of them is limited. Requests will be filled in order of receipt.

A deduction of $2 per volume will be made for any volume out of print when the order is received.

SEARCHES IN THE LIBRARY

As the Library of the American Society of Civil Engineers has been merged in the Engineering Societies Library, requests for searches, copies, translations, etc., should be addressed to the Director, Engineering Societies Library, 29 West 39th Street, New York City, who will gladly give information concerning the charges for the various kinds of service. A more comprehensive statement in regard to this matter will be found on page 21 of the Year Book for 1921.

PAPERS AND DISCUSSIONS

Members and others who take part in the oral discussions of the papers presented are urged to revise their remarks promptly. Written communications from those who cannot attend the meetings should be sent in at the earliest possible date after the issue of a paper. Written discussion on a given paper will be closed three months after the paper has been published, so that the author's closure can be printed four months after the paper.

All manuscripts submitted for publication should preferably be typewritten, and always double spaced. Drawings and diagrams should be on separate sheets, drawn to a scale suitable for about one-half to one-fourth reduction.

All papers accepted by the Publication Committee are classified by the Committee with respect to their availablity for discussion at meetings.

Papers which, from their general nature, appear to be of a character suitable for oral discussion, will be set down for presentation to a future meeting of the Society, and, on these, oral discussions, as well as written communications, will be solicited.

All papers which do not come under this heading, that is to say, those which from their mathematical or technical nature, in the opinion of the Committee, are not adapted to oral discussion, will not be scheduled for presentation to any meeting. Such papers will be published in the same manner as those which are to be presented at meetings, but written discussions only will be requested for subsequent publication in *Proceedings* and with the paper in the volumes of *Transactions*.

The Board of Direction has adopted rules for the preparation and presentation of papers, which will be found on page 36 of the Year Book for 1921.

LOCAL SECTIONS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

San Francisco Section (Constitution Approved by Board, 1905).

Frederick R. Muhs, President; H. D. Dewell, Secretary-Treasurer, 503 Market Street, San Francisco, Cal.

Bi-monthly meetings are held at 6 P. M., at the Engineers' Club, 57 Post Street, on the third Tuesday of February, April, June, August, October, and December, the last being the Annual Meeting. Informal luncheons are held at noon, every Wednesday, at the Engineers' Club. All members of the Society will be gladly welcomed.

Colorado Section (Constitution Approved by Board, 1909).

A. N. Miller, President; Walter L. Drager, Secretary-Treasurer, 412 Tramway Building, Denver, Colo.

Meetings are held on the second Monday of each month, except July and August, usually preceded by an informal dinner. Weekly luncheons are held on Wednesday, at 12.30 P. M., at Daniels and Fisher's. Visiting members of the Society are urged to attend.

Atlanta Section (Constitution Approved by Board, 1912).

J. T. Wardlaw, President; R. S. Fiske, Secretary-Treasurer, 1530 Healey Building, Atlanta, Ga.

Informal luncheons are held on the second Tuesday of each month, at 1.00 P. M., at the Ansley Hotel, to which visiting members of the Society are welcome. Visitors desiring information will telephone the Secretary, "Ivy, 3605."

Baltimore Section (Constitution Approved by Board, 1914).

Ezra B. Whitman, President; George S. Robertson, Sr., Secretary-Treasurer, 1628 Linden Avenue, Baltimore, Md.

Buffalo Section (Constitution Approved by Board, 1921).

A. L. Johnson, President; Bruce L. Cushing, Secretary-Treasurer, 80 West Genesee Street, Buffalo, N. Y.

Central Ohio Section (Constitution Approved by Board, 1921).

F. H. Eno, President; H. D. Bruning, Secretary, 935 Madison Avenue, Columbus, Ohio.

Meetings are held at the rooms of the Engineers' Club of Columbus in the Southern Hotel. The Annual Meeting is held on the second Friday of November and at least two other meetings are held each year the dates of which are designated by the Board of Direction of the Section.

Cincinnati Section (Constitution Approved by Board, 1920).

Edgar Dow Gilman, President; Alphonse M. Westenhoff, Secretary, 13 East Third Street, Cincinnati, Ohio.

Cleveland Section (Constitution Approved by Board, 1915).

J. E. A. Moore, President; George H. Tinker, Secretary-Treasurer, 516 Columbia Building, Cleveland, Ohio.

Regular meetings are held on the second Wednesday of each month, at 12.15 P. M., in the rooms of the Section, Hotel Winton. Luncheon is served, and all visiting members of the Society are invited to attend.

Connecticut Section (Constitution Approved by Board, 1919).

Charles Rufus Harte, President; Clarence M. Blair, Secretary-Treasurer, 785 Edgewood Avenue, New Haven, Conn.

The Annual Meeting is held in April; fortnightly meetings alternate between Hartford and New Haven, Conn. These meetings are informal luncheon gatherings, held usually at noon on Saturday. Members are privileged to invite guests regardless of their affiliation as engineers.

Detroit Section (Constitution Approved by Board, 1916).

David A. Molitor, President; Dalton R. Wells, Secretary-Treasurer, 624 McKerchey Building, Detroit, Mich.

Regular meetings are held on the second Friday of December, April, and October, the last being the Annual Meeting.

District of Columbia Section (Constitution Approved by Board, 1916).

John C. Hoyt, President; James H. Van Wagenen, Secretary-Treasurer, 2001 Sixteenth Street, N. W., Washington, D. C.

Duluth Section (Constitution Approved by Board, 1917).

John L. Pickles, President; Walter G. Zimmermann, Secretary, 203 Wolvin Building, Duluth, Minn.

Regular meetings are held at noon on the third Monday of each month, usually at the Kitchi Gammi Club, to which visiting members of the Society will be welcomed. The Annual Meeting is held on the third Monday in May.

Illinois Section (Constitution Approved by Board, 1916).

Charles B. Burdick, President; W. D. Gerber, Secretary-Treasurer, 913 Chamber of Commerce, Chicago, Ill.

Regular meetings are held on the second Monday of March, June, September, and December, the last being the Annual Meeting.

Iowa Section (Constitution Approved by Board, 1920).

J. H. Dunlap, President; R. W. Crum, Secretary, Care, Iowa State Highway Commission, Ames, Iowa.

Kansas City (Mo.) Section (Constitution Approved by Board, 1921).

Alexander Maitland, Jr., President; Henry C. Tammen, Secretary-Treasurer, 1012 Baltimore Avenue, Kansas City, Mo.

Regular meetings of the Section are held on the first Tuesday of March, June, September, and December, the last being the Annual Meeting. The members of the Kansas City Engineers' Club meet at luncheon at the University Club every Tuesday from 12 M. to 2 P. M., and all members of the Society are invited to attend these luncheons.

Kansas Section (Constitution Approved by Board, 1920).

L. E. Conrad, President; Frank S. Altman, Secretary-Treasurer, 1114 Garfield Avenue, Topeka, Kans.

Los Angeles Section (Constitution Approved by Board, 1913).

Ralph J. Reed, President; Floyd G. Dessery, Secretary, 618 Central Building, Los Angeles, Cal.

Regular monthly meetings are held on the second Wednesday of each month, the Annual Meeting in December. Informal luncheons in connection with the Joint Technical Societies of Los Angeles are held at 12.15 P. M., every Thursday at the Broadway Department Store Café.

Louisiana Section (Constitution Approved by Board, 1914).

Ole K. Olsen, President; F. A. Muth, Secretary, 224 Custom House Building, New Orleans, La.

Regular meetings are held at The Cabildo, New Orleans, La., on the first Monday of January, April, July, and October.

Nashville Section (Constitution Approved by Board, 1921).

Arthur J. Dyer, President; Granbery Jackson, Secretary-Treasurer, 220 Capitol Boulevard, Nashville, Tenn.

Nebraska Section (Constitution Approved by Board, 1917).

Rodman M. Brown, President; Homer V. Knouse, Secretary-Treasurer, 200 City Hall, Omaha, Nebr.

Regular meetings are held on the first Saturday of each month, except July and August. The Annual Meeting is held in Lincoln, Nebr., on the second Friday in January. Visiting members of the Society are especially urged to communicate with the Secretary when in the city.

New York Section (Constitution Approved by Board, 1920).

Nelson P. Lewis, President; J. P. J. Williams, Secretary, 33 West 39th Street, New York City.

Regular meetings are held in the Engineering Societies Building, 29 West 39th Street, New York City, on the third Wednesday of each month, except January and the Annual Meeting in May, held on the second Wednesday of the month.

Northeastern Section (Constitution Approved by Board, 1921).

Frank B. Sanborn, Chairman; Charles W. Banks, Secretary, Wentworth Institute, Boston, Mass.

Northwestern Section (Constitution Approved by Board, 1914).

Charles L. Pillsbury, President; Paul C. Gauger, Secretary, 945 Osceola Avenue, St. Paul, Minn.

Meetings are held bi-monthly, alternating between St. Paul and Minneapolis, on the third Friday of each month.

Oklahoma Section (Constitution Approved by Board, 1920).

Max L. Cunningham, President; R. E. Brownell, Secretary-Treasurer, 402 First National Bank Building, Oklahoma, Okla.

Philadelphia Section (Constitution Approved by Board, 1913).

John Meigs, President; S. C. Hollister, Secretary, 1200 Land Title Building, Philadelphia, Pa.

Regular meetings are held at the Engineers' Club on the first Monday in January, April, and October, the last being the Annual Meeting. Special meetings are also held at times announced in advance.

Pittsburgh Section (Constitution Approved by Board, 1918).

N. S. Sprague, President; Nathan Schein, Secretary-Treasurer, 1510 Carson Street, Pittsburgh, Pa.

Portland (Ore.) Section (Constitution Approved by Board, 1913).

M. E. Reed, President; C. P. Keyser, Secretary, 318 City Hall, Portland, Ore.

Meetings are held regularly on the third Friday of each month. All members of the Society in any grade are cordially invited to attend.

Providence (R. I.) Section (Constitution Approved by Board, 1920).

Sydney Wilmot, Chairman; Robert L. Bowen, Secretary-Treasurer, 26 Sycamore Street, Providence, R. I.

The Section regularly holds meetings jointly with the Structural and Municipal Sections of the Providence Engineering Society, at the Society Rooms, 29 Waterman Street, on the fourth Tuesday of each month, from September to May. The Annual Meeting is held in May. All visiting members of the Society are cordially invited to attend these meetings.

St. Louis Section (Constitution Approved by Board, 1914).

E. B. Fay, President; William C. E. Becker, Secretary-Treasurer, 426 City Hall, St. Louis, Mo.

The Annual Meeting is held on the fourth Monday in November. Two meetings each year for the presentation and discussion of technical papers are held in the Auditorium of the Engineers' Club, and are open to members of the Associated Societies. Other "get-together" meetings are held regularly for dinner or luncheon on the fourth Monday of each month except July, August, and November.

San Diego Section (Constitution Approved by Board, 1915).

F. J. Grumm, President; J. Y. Jewett, Secretary-Treasurer, Administration Building, Balboa Park, San Diego, Cal.

Regular meetings are held on the third Tuesday of each month at the Chamber of Commerce. Visiting members of the Society are invited to attend.

Seattle Section (Constitution Approved by Board, 1913).

T. E. Phipps, President; Frank H. Fowler, Secretary-Treasurer, 1319 L. C. Smith Building, Seattle, Wash.

Regular meetings, with luncheon, are held at the Engineers' Club, on the last Monday of each month. All members in any grade of the Society are cordially invited to attend, and if located in this District for any length of time, their membership in the Section will be appreciated.

Spokane Section (Constitution Approved by Board, 1914).

E. G. Taber, President; Charles E. Davis, Secretary-Treasurer, 401 City Hall, Spokane, Wash.

Meetings are held on the second Friday of each month. These meetings are noonday luncheons at Davenport's, and all visiting members of the Society are invited to attend.

Texas Section (Constitution Approved by Board, 1913).

E. B. Cushing, President; E. N. Noyes, Secretary, 1107 Dallas County Bank Building, Dallas, Tex.

Utah Section (Constitution Approved by Board, 1916).

W. R. Armstrong, President; H. S. Kleinschmidt, Secretary-Treasurer, 222 Felt Building, Salt Lake City, Utah.

The Annual Meeting is held on the first Wednesday in April. The time of other meetings is not fixed, but this information will be furnished on application to the Secretary.

STUDENT CHAPTERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS*

Leland Stanford, Jr., University Student Chapter, Organized 1920.

R. L. Wing, President; John H. Colton, Corresponding Secretary, Box 121, Stanford, Cal.

Alabama Polytechnic Institute Student Chapter, Organized 1921.

Alfred D. Boyd, Secretary, Alabama Polytechnic Institute, Auburn, Ala.

Braune Civil Engineering Society (University of Cincinnati) Student Chapter, Organized 1920.

John W. Guilday, President; C. A. Harrell, Secretary of Section 10; R. Blickensderfer, Secretary of Section 20; University of Cincinnati, Cincinnati, Ohio.

California Institute of Technology Student Chapter, Organized 1921.

J. Arthur Macdonald, Secretary, California Institute of Technology, Pasadena, Cal

Civil Engineering Society of Rensselaer Polytechnic Institute Student Chapter, Organized 1920.

William Minot Thomas, President; Earl D. Hopkins, Secretary, 147 Eighth Street, Troy, N. Y.

Cornell University Student Chapter, Organized 1921.

John J. Chavanne, Jr., Secretary, Cornell University, Ithaca, N. Y.

Drexel Institute Student Chapter, Organized 1920.

C. V. Nishwitz, Chairman; Raymond Radbill, Secretary, Drexel Institute, Philadelphia, Pa.

Iowa State College Student Chapter, Organized 1920.

Alfred W. Warren, Secretary, Iowa State College, Ames, Iowa.

Johns Hopkins University Student Chapter, Organized 1921.

Eric M. Arndt, President; Melvin E. Scheidt, Secretary, Box 566, Homewood, Baltimore, Md.

Massachusetts Institute of Technology Student Chapter, Organized 1921.

D. H. McCreery, President; T. S. Wray, Secretary, Massachusetts Institute of Technology, Cambridge, Mass.

New York University Student Chapter, Organized 1921.

William J. Kiehnle, President; George H. Martin, Jr., Secretary, New York University, University Heights, New York City.

Oregon State Agricultural College Student Chapter, Organized 1921.

John B. Alexander, Secretary, Omega Upsilon House, Oregon State Agricultural College, Corvallis, Ore.

Pennsylvania State College Student Chapter, Organized 1920.

Arthur H. McFadden, President; William W. Seltzer, Secretary, Pennsylvania State College, State College, Pa.

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* By a recent ruling of the Board of Direction, the minimum membership of a Student Chapter has been fixed at 12 instead of 20.

Polytechnic Institute of Brooklyn Student Chapter, Organized 1921.

Richard Kanegsberg, Secretary, Polytechnic Institute of Brooklyn, Brooklyn, N. Y.

Purdue University Student Chapter, Organized 1921.

Donald A. Leach, President, 208 Fowler Avenue, West Lafayette, Ind.

Rose Polytechnic Institute Student Chapter, Organized 1921.

Kenneth L. De Blois, President; Duncan Baker, Secretary, 1606 North Eighth Street, Terre Haute, Ind.

Rutgers College Student Chapter, Organized 1921.

L. C. Kuhl, President; A. C. Ely, Secretary, 105 Winants Hall, Rutgers College, New Brunswick, N. J.

State University of Iowa Student Chapter, Organized 1921.

C. E. Stickney, Secretary, State University of Iowa, Iowa City, Iowa.

Swarthmore College Student Chapter, Organized 1921.

Frank Lemke, President; H. Chandler Turner, Jr., Secretary, Swarthmore College, Swarthmore, Pa.

Syracuse University Student Chapter, Organized 1921.

Arthur V. Dollard, Secretary, College of Applied Science, Syracuse University, Syracuse, N. Y.

University of California Student Chapter, Organized 1921.

H. G. Gerdes, Secretary, Care, Prof. Charles Derleth, Jr., College of Civil Engineering, University of California, Berkeley, Cal.

University of Colorado Civil Engineering Society Student Chapter, Organized 1920.

Herbert Altvater, President; Charles Bowden, Secretary, 1229 University Avenue, Boulder, Colo.

University of Illinois Student Chapter, Organized 1921.

A. L. R. Sanders, President; M. E. Jansson, Secretary, University of Illinois, Urbana, Ill.

University of Kansas Student Chapter, Organized 1921.

Waldo G. Bowman, Secretary, 1106 Ohio Street, Lawrence, Kans.

University of Kentucky Student Chapter, Organized 1921.

B. O. Bartee, Secretary, University of Kentucky, Lexington, Ky.

University of Maine Student Chapter, Organized 1921.

George H. Ferguson, Jr., Secretary, University of Maine, Orono, Me.

University of Minnesota Student Chapter, Organized 1921.

C. L. Swanson, President, 1716 Tyler Street, N. E., Minneapolis, Minn.

University of Nebraska Student Chapter, Organized 1921.

J. E. Applegate, President; W. H. Mengel, Secretary, University of Nebraska, Lincoln, Nebr.

University of Pennsylvania Student Chapter, Organized 1920.

Charles W. Foppert, President; Fred Welchy, Secretary, University of Pennsylvania, Philadelphia, Pa.

University of Pittsburgh Student Chapter, Organized 1921.

L. W. Fletcher, President; J. M. Daniels, Secretary, University of Pittsburgh, Pittsburgh, Pa.

University of Texas Student Chapter, Organized 1921.

W. H. D. Taylor, President; Phil M. Ferguson, Secretary, 2505 Guadalupe Street, Austin, Tex.

University of Washington Student Chapter, Organized 1921.

G. B. Richardson, President; G. E. Large, Secretary, 4518 Eleventh Avenue, N. E., Seattle, Wash.

University of Wisconsin Student Chapter, Organized 1921.

Herbert Wheaton, President; Olaf N. Rove, Secretary, University of Wisconsin, Madison, Wis.

Virginia Military Institute Student Chapter, Organized 1921.

Benjamin F. Parrott, Secretary, Virginia Military Institute, Lexington, Va.

Washington University Collimation Club Student Chapter, Organized 1920.

William D. Rolfe, President; Erwin Bloss, Secretary, Washington University, St. Louis, Mo.

West Virginia University Student Chapter, Organized 1921.

J. E. Wheeler, President; Milton Jarrell, Secretary, 113 Beverly Avenue, Morgantown, W. Va.

Yale University Student Chapter, Organized 1921.

W. S. Moore, President; T. T. McCrosky, Secretary, Sheffield Scientific School, Yale University, New Haven, Conn.

PRIVILEGES OF ENGINEERING SOCIETIES EXTENDED TO MEMBERS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

Members of the American Society of Civil Engineers will be welcome in the Reading Rooms and at the meetings of many engineering societies in all parts of the world. A list of such societies will be found on pages 48, 49, and 50 of the Year Book of the Society for 1921.

NEW BOOKS*

(From November 1st to November 30th, 1921)

The statements made in these notices are taken from the books themselves, and this Society is not responsible for them.

DONATIONS TO ENGINEERING SOCIETIES LIBRARY

DIAGNOSING OF TROUBLES IN ELECTRICAL MACHINES.

By Miles Walker. Lond. and N. Y., Longmans, Green and Co., 1921. 450 pp., diagrams, 10 x 7 in., cloth. $10.50.

During the last thirty years the author of this book has had a great many troubles in connection with electrical machinery brought to his notice, many of which were difficult to diagnose and correct. He here attempts to record his experience in logical order, to assist others when dealing with similar troubles. The book discusses troubles due to defective insulation, over-heating, low efficiency, and those peculiar to alternating and direct-current generators and motors, synchronous converters, motor generators, and induction motors. It is especially concerned with troubles in the field, not with factory tests.

CENTRAL STATION RATES IN THEORY AND PRACTICE.

By H. E. Eisenmenger. Chic., Frederick J. Drake & Co., 1921. 382 pp., illus., 7 x 5 in., fabrikoid.

A textbook for students of electric rates, intended to meet the needs of both beginners and experts. Discusses the cost of electric service, its price, systems of charging, rate analysis, the accuracy of rates, and public regulation of public utilities. Appeared serially in the *Electrical Review.*

STEAM ENGINE VALVES AND VALVE GEARS.

By E. L. Ahrons. (Technical Primers.) Lond. and N. Y., Sir Isaac Pitman & Sons, Ltd., 1921. 112 pp., illus., 6 x 4 in., cloth. 85 cents.

This primer gives, in an elementary form, a concise description of the usual forms of steam engine valves and valve gears, and an explanation of their action.

STEAM ROAD VEHICLES.

By L. M. Meyrick-Jones. Second Edition. Lond., Iliffe & Sons, Ltd. 213 pp., illus., 8 x 6 in., cloth. 5 Shillings.

This book is intended to provide an explanation of the theory and practice of steam road transport, suited to the needs of owners and as an instruction book for drivers and mechanics. Its twenty-eight chapters describe the principles involved in the generation of steam and the construction of the various units that make up the vehicle. This edition has been revised and enlarged. The practice described is exclusively British.

MECHANICAL PRINCIPLES OF THE AEROPLANE.

By S. Brodetsky. N. Y., The Macmillan Co., 1921. 272 pp., illus., 10 x 6 in., cloth. $7.00.

Section One, "Motion in Air", contains a brief statement of the general theory of resisted motion and a discussion of the dynamics of a body moving in air, and the theory of steady flight, which indicates what it is that aerodynamical research must investigate. In Section Two, "Dynamics of Air", the hydrodynamical method of attacking the problem is set forth, and in Section Three, "Aeroplane Motion", theoretical and experimental results are applied to the steady flight and stability of æroplanes, followed by an account of the motion of a disturbed æroplane. The treatment is confined to the mathematical parts of these problems.

LES COMBUSTIBLES LIQUIDES ET LEURS APPLICATIONS.

By the Syndicat d'Applications Industrielles des Combustibles Liquides. Paris, Gauthier-Villars et Cie., 1921. 621 pp., illus., 6 x 4 in., cloth.

This handbook, published by an association of French companies interested in the production and use of liquid fuel, has been prepared as a practical guide to users and dealers It includes the regulations governing the importation and use of liquid fuels, insurance laws, brief descriptions of the chief oil-producing countries, the principal fuel oils and lubricants, and methods for testing them. Descriptions of the leading French types of internal combustion engines, furnaces, and boilers are given, as well as directions for storing and shipping oil. The final section consists of conversion tables and coefficients.

* Unless otherwise specified, books in this list have been donated by the publishers.

POWER'S PRACTICAL REFRIGERATION.

Compiled by the Editorial Staff of *Power*. N. Y. and Lond., McGraw-Hill Book Co., Inc., 1921. 283 pp., illus., tab., 8 x 6 in., fabrikoid. $2.00.

A volume dealing with the practice of refrigeration, but also including the laws governing its production. Chiefly made up of articles that have appeared in *Power* and have proven of particular value to those operating refrigerating plants.

MECHANICAL HANDLING OF GOODS.

By C. H. Woodfield. (Technical Primers.) Lond. and N. Y., Sir Isaac Pitman & Sons, Ltd., 1921. 116 pp., illus., 6 x 4 in., cloth. $.85.

The object of this book is to set forth sufficient information on the handling of goods and material to enable the uninitiated to understand the methods and equipment used and appreciate the economic possibilities of dealing with goods by mechanical methods.

CONDENSED CATALOGUES OF MECHANICAL EQUIPMENT.

Published by the American Society of Mechanical Engineers. Eleventh Annual Volume, October, 1921. 932 pp., illus., 9 x 6 in., cloth. $4.00.

The eleventh issue of this convenient collection of commercial data on mechanical equipment and the accompanying directories of manufacturers and consulting engineers follows the form of preceding editions. It has, however, been enlarged considerably and revised carefully. The firms listed number 4 000, under 3 000 classes, and 495 of these have published data about their products in the book. The number of consulting engineers is 1 000, classified under 400 lines of specialization.

DIE FÖRDERUNG VON MASSENGUTERN; VOL. 1

By Georg von Hanffstengel. Berlin, Julius Springer, 1921. 306 pp., illus., 9 x 6 in., cloth. 234 marks.

This volume of the third edition of this useful treatise deals with belt, chain, bucket, screw, spiral, pneumatic, and hydraulic conveyors, together with some minor types. The text is practical as well as theoretical, and covers modern practice very thoroughly. The work has been thoroughly revised.

MACHINE DRAWING.

By Carl L. Svensen. N. Y., D. Van Nostrand Co., 1921. 214 pp., illus., 9 x 6 in., cloth. $2.25

This textbook is intended for students who have had previous instruction in mechanical drawing and is intended to develop an understanding of the relation of machine drawing to engineering. It includes a complete treatment of working drawings, drafting-room practice, a chapter on the principles and practice of dimensioning, a study of the common machine details, jigs, and fixtures, and a large collection of problems.

BLEACHING.

By S. H. Higgins. (Publications of the University of Manchester, No. 142.) Manchester, University Press; Lond. and N. Y., Longmans, Green & Co., 1921. 137 pp., 9 x 6 in., cloth. $3.75.

The idea of this volume is not to give an account of the subject of bleaching, but to act as a supplement to other books, of which there are many, dealing with this industry. The author's intention has been to discuss the important researches of recent years bearing on bleaching as a basis for further research.

NOS USINES METALLURGIQUES DEVASTEES, 1914-1918.

Paris, La Revue de Métallurgie, 1921. 233 pp., illus., 11 x 9 in., paper. 25 francs.

This book has been prepared as a memorial of the ruin wrought on the metallurgical works of France by the invading Germans. An introduction by Prof. Léon Guillet describes the general losses of the country, the influence of the invasion on the metallurgical industry, and the reaction of French metallurgists to the challenge. Other engineers report in detail on the condition of eighteen of the most important plants before the World War and when returned after the armistice. Three hundred photographs accompany these reports and show, more vividly than words, the thoroughness of the destruction. The volume is a most interesting record of depredation.

CHEMICAL WARFARE.

By Amos A. Fries and Clarence J. West. N. Y. and Lond., McGraw-Hill Book Co., Inc., 1921. 445 pp., illus., 8 x 6 in., cloth. $3.50.

The story of the Chemical Warfare Service of the Army during the World War, written to serve as a history of that service and also to serve as a textbook covering the fundamental facts, for the Army, the reserve officer, and the chemist.

OIL-FIELD PRACTICE.

By Dorsey Hager. N. Y. and Lond., McGraw-Hill Book Co., Inc., 1921. 310 pp., illus., diagrams, 7 x 5 in., fabrikoid. $3.00.

The subjects treated in this volume are the acquisition of lands, development drilling, development production methods, transportation, storage, fires, avoidable wastes and losses, refining methods, valuation and buying, and general observations on the industry. The Appendix contains sample forms for records, useful tables, and a glossary. The book presents American methods of developing oil properties and is intended to give an intelligent insight into the petroleum industry as a whole. To a certain extent it supplements the author's earlier work, "Practical Oil Geology".

MINING PHYSICS AND CHEMISTRY.

By J. W. Whitaker. Lond., Edward Arnold & Co., 1921. 268 pp., diagrams, tab., 7 x 5 in., cloth. $3.00. (Gift of Longmans, Green & Co.)

This introductory textbook includes the chemical and physical knowledge needed by an underground miner or official, particularly those working in collieries. Questions of ventilation, spontaneous ignition, explosions and explosives, waters, mine gases, and other subjects of like importance are treated in simple language.

HANDBOOK FOR FIELD GEOLOGISTS.

By C. W. Hayes. Third Edition, Revised and Rearranged by Sidney Paige. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1921. 166 pp., illus., 7 x 5 in., fabrikoid. $2.50.

This manual, of convenient pocket size, contains a concise summary of the methods and instruments that experience has shown to be most useful in field work, together with outlines and schedules covering investigations in the several fields of geology, such as the interpretation of land forms, glaciers, and glacial deposits, metalliferous deposits, etc. This edition has been revised and an appendix on mineralogy has been added.

MANUAL OF DETERMINATIVE MINERALOGY.

By Charles H. Warren. N. Y. and Lond., McGraw-Hill Book Co., Inc., 1921. 163 pp., 7 x 5 in., fabrikoid. $2.00.

This manual has been prepared to enable the student to supplement his descriptive text and crystallography with a relatively inexpensive, but satisfactory determinative text. It represents the course given in the Massachusetts Institute of Technology, and has been thoroughly tested by use for several years.

ELEKTRISCHE FÖRDERMASCHINEN.

By W. Philippi. Leipzig, S. Hirzel, 1921. 304 pp., illus., 9 x 6 in., paper.

The use of electric hoisting machinery in mining is covered from several viewpoints, mechanical, electrical, and economic, with special stress on the last aspect of the question. The question, whether electric hoisting shall be adopted for a given mine, is the one most discussed.

COKE-OVEN AND BY-PRODUCT WORKS CHEMISTRY.

By Thomas Biddulph-Smith. Lond., Charles Griffin & Co., Ltd.; Phila., J. B. Lippincott Co., 1921. 180 pp., illus., pl., 9 x 6 in., cloth. $7.00.

This concise manual of methods of chemical analysis is intended to cover the general work required for the chemical control of by-product coking plants, and is designed for works managers and chemists. The value of the book is enhanced by an appendix giving a general survey of the nature of the more valuable and also the lesser known substances obtainable by distilling coal tar.

EARLY SCIENCE IN OXFORD: PART 1: CHEMISTRY.

By R. T. Gunther. Lond., Hazell, Watson and Viney, Ltd., 1920. 91 pp., illus., pl., 8 x 6 in., paper. $3.50. (Gift of Oxford University Press, American Branch.)

This account of early study of chemistry at Oxford traces the story from its beginnings, with Roger Bacon in 1214, down to the early Nineteenth Century. It is the first attempt to bring together such scattered information as is relevant to a fuller history of the progress of science, and will be followed by parts treating of other branches. An interesting account is given of early Oxford chemists, of their laboratories, and their apparatus.

FOUNDATIONS OF CHEMICAL THEORY.

By R. M. Caven. N. Y., D. Van Nostrand Co., 1921. 266 pp., diagrs., 9 x 6 in., cloth. $4.00.

This work endeavors to disclose the foundations of chemistry and make them plain and real to the average student, without bewildering him by including the multitudinous array of facts presented by the science. The author hopes that the book will also give the general reader a satisfactory account of the meaning of modern chemistry.

CHEMICAL DISINFECTION AND STERILIZATION.

By Samuel Rideal and E. K. Rideal. Lond., Edward Arnold & Co., 1921. 313 pp., 9 x 6 in., cloth. $7.50. (Gift of Longmans, Green and Co.)

This book collects and summarizes some of the more important applications of general methods for the sterilization of food, air, and water, for public and private disinfection, for extirpating non-bacterial parasites, and for preserving wood. It also describes the various chemicals used in disinfection, and methods for their analysis and testing. Numerous bibliographies are included, forming a useful brief summary of the scattered literature of a wide field.

ORGANIC SYNTHESES; Vol. 1.

Roger Adams, Editor-in-Chief. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1921. 84 pp., illus., 9 x 6 in., cloth. $1.50.

The first of a series of annual volumes devoted to the publication of accurate, detailed descriptions of the most convenient laboratory methods for preparing organic chemicals in small lots. The chemicals included will be selected from those needed in research laboratories. The series is designed to overcome the difficulty experienced by chemists during and since the war, when the less common chemicals formerly obtained from Germany can no longer be had, except at prohibitive prices.

ELECTRONS AND ETHER WAVES,

Being the Twenty-Third Robert Boyle Lecture. By Sir William Bragg. Lond. and N. Y., Oxford Univ. Press, 1921. 14 pp. 9 x 6 in., paper. 45 cents.

This address is concerned with one of the outstanding problems in physics, the connection between ether waves and electrons and the relation between the wave length of the ether radiations and the velocity of the ejected electrons. The lecture gives a non-mathematical account of present information on the matter.

PHYSICAL PROPERTIES OF COLLOIDAL SOLUTIONS.

By E. F. Burton. (Monographs in Physics.) Second Edition. Lond. and N. Y., Longmans, Green and Co., 1921. 221 pp., illus., 9 x 6 in., cloth. $4.25.

This outline of the study of colloidal solutions has to do particularly with their relation to the development of physics. For this reason an extended treatment is given of the development of the ultra-microscope and the confirmation of the kinetic theory of matter by the Brownian movement. In this new edition the book has been thoroughly revised and partly rewritten.

GRAPHICAL METHODS.

By William C. Marshall. N. Y. and Lond., McGraw-Hill Book Co., Inc., 1921. 253 pp., charts, 9 x 6 in., cloth. $3.00.

A general treatise on the construction and use of graphical charts. Includes charts intended to appeal to the general public, those of interest to executives, and those intended to facilitate engineering and scientific calculations. Gives many examples of the application of charts to a great variety of purposes and contains an extensive bibliography of published charts.

ELEMENTS D'ANALYSE MATHEMATIQUE.

By Paul Appell. Fourth Edition. Paris, Gauthier-Villars et Cie., 1921. 715 pp., 10 x 7 in., paper. 65 fr.

This textbook of the elements of mathematical analysis pays particular attention to the use of analysis in geometry, physics, and mechanics, and is intended for engineers and physicists. Numerous examples of the applications of analysis are included, and all theories are illustrated by application to particular cases. This edition has been revised throughout and extended considerably.

ANNALS OF THE AMERICAN ACADEMY OF POLITICAL AND SOCIAL SCIENCE,

May, 1920; Vol. 89, No. 178. 289 pp., 9 x 6 in., paper. $1.25.

This volume of essays discusses the economic significance of present-day prices, price factors in typical commodities, wages, profits and excess profits taxes, products, co-operation, international finance and trade in their relation to prices, inflation and prices, and the world's monetary problems. The papers included are by well-known economists, business men, and engineers.

EMPLOYMENT MANAGEMENT, WAGE SYSTEMS AND RATE SETTING.

N. Y., The Industrial Press, 1921. 103 pp., 9 x 6 in., paper. $1.00.

This concise description of systematic methods of employing and placing men, and of wage payment systems, is based on articles that have appeared in *Machinery*, describing the practice in the Westinghouse Electric and Manufacturing Company, R. K. LeBlond Machine Tool Company, Norton Company, and other manufacturing plants.

PROFIT-SHARING BY AMERICAN EMPLOYERS:

A Report of the Profit-Sharing Department of the National Civic Federation. N. Y., E. P. Dutton & Co., 1921. 416 pp., 8 x 5 in., cloth. $8.00.

The first edition of this book, published in 1916 by The National Civic Federation, was based on analyses of more than two hundred plans for profit-sharing in use in the United States. It was intended to present testimony, from managers of manufacturing establishments, concerning the success and failure of such plans. This edition is practically a new book, as much of the former material has been omitted and numerous additions have been made, enlarging the book and bringing it up to 1919.

WASTE IN INDUSTRY.

By the Committee on Elimination of Waste in Industry of the Federated American Engineering Societies. N. Y. and Lond., McGraw-Hill Book Co., Inc., 1921. 409 pp., charts, tab., 9 x 6 in., cloth. $4.00.

This committee of seventeen engineers was appointed in January, 1921, by Herbert C. Hoover, M. Am. Soc. C. E., first President of the Federated American Engineering Societies. It was instructed to gather quickly concrete information that might stimulate action on the elimination of waste and lay the foundation for further study. The present report is an analysis of waste in six typical branches of industry (building industry, men's clothing manufacturing, shoe manufacturing, printing, metal trades, textile manufacturing), based on five months of intensive study, carefully planned, and rapidly executed. It discloses losses and waste due to the restraint and dissipation of the creative power of those who work in industry, and presents for the first time a collective endorsement of a general analysis of the sources and causes of waste and recommendations for its elimination.

COTTON INDUSTRY IN FRANCE.

By R. B. Forrester, Lond., and N. Y., Longmans, Green & Co., 1921. 142 pp., map, 9 x 6 in., cloth. $3.75.

This study was undertaken to provide an account of the conditions prevailing in the industry in France, to point out the characteristic features of recent development, and to contrast its position with that of the cotton industry in other countries. The material having been collected in 1910, 1911, and 1912, the report is essentially a picture of the conditions existing in 1914, but a discussion of post-war conditions is appended.

FOREST MENSURATION.

By Herman Haupt Chapman. N. Y., John Wiley & Sons, Inc.; Lond., Chapman & Hall, Ltd., 1921. 553 pp., illus., 9 x 6 in., cloth. $5.00.

This text is a thorough discussion of the measurement of the volume of felled timber in the form of logs or other products, of the volume of standing timber, and of the growth of trees, stands of timber, and forests. It is designed for students, purchasers, and owners of timber lands and timber operators. The work, the author states, is the successor of Graves' "Forest Mensuration", but is an entirely new presentation of the subject, not a revision of that book.

INVENTION THE MASTER-KEY TO PROGRESS.

By Bradley A. Fiske. N. Y., E. P. Dutton & Co., 1921, 356 pp., illus., 8 x 6 in., cloth. $4.00

The thesis of this author is that invention, acting through literature, science, art, war, and the other activities of men, has initiated all creative human progress; and his book is an interesting account of what inventors have accomplished through the ages, and a forecast of what may be done in the future, if the art of invention is properly fostered.

SPECIAL LIBRARIES DIRECTORY.

Edited by Dorsey W. Hyde, Jr. Wash., Special Libraries Assoc., 1921. 123 pp., 9 x 6 in., paper. $2.00.

This directory is a comprehensive survey of the specialized collections of literature on various subjects in the United States. More than 1 300 libraries belonging to universities, societies, business houses, and other agencies are listed, with their location, rules for use, and brief accounts of their resources. The list is arranged by subject and also geographically. Many of these libraries are concerned with engineering and allied subjects. The list will prove valuable to research students, in indicating sources of information.

RAILWAY SIGNALING.

By Everett Edgar King. N. Y. and Lond., McGraw-Hill Book Co., Inc., 1921. 371 pp., illus., diagrams, 9 x 6 in., cloth. $4.00.

The purpose of this book is to collect what is already in common practice in railway signaling and to present it in textbook form, suitable for use by those beginning the study of this subject. The volume discusses signal indications, interlocking, block signaling, signal mechanisms, and highway-crossing signals.

CENTRIFUGAL PUMPS.

By J. W. Cameron. Lond., Scott, Greenwood & Son, 1921. 142 pp., illus., 9 x 6 in., cloth. $3.75. (Gift of D. Van Nostrand Co.)

This small book discusses the theory of these pumps, hydraulic losses, hydraulic efficiency, bearings, effect of vane angle on efficiency, pump details, axial thrust balancing, calculation and design of pumps, and commercial types. The work is intended for engineers and draftsmen.

VORUNTERSUCHUNG UND BERECHNUNG DER GRUNDWASSERFASSUNGSANLAGEN.

By J. Versluys. München, R. Oldenbourg, 1921. 40 pp., 9 x 5 in., paper. 7.50 marks.

The author of this pamphlet was in charge, from 1915 to 1919, of the hydrological investigations and calculations relating to the potable water supply carried on by the Government of Holland. The procedures adopted have been presented in various reports published in the Dutch language, and he here presents, in brief form, the principles on which the work was based. These principles are directly applicable to the problem which the hydrologist has to consider in the storage of ground-water, and outlines are given for the more usual calculations.

PORT OF NEW YORK ANNUAL.

Compiled and Edited by Alexander R. Smith, N. Y., Smith's Port Publishing Co., Inc., 1920. 526 pp., illus., 11 x 8 in., cloth. $5.00.

The information here presented will be of interest to merchants and shippers generally, for it covers the port of New York in a large way, and discusses its problems from many angles. The present and proposed facilities in the various Boroughs of New York and in Newark and Jersey City are described with considerable detail. Information is also given on railroad and canal connections to the port, on the Hudson River vehicular tunnel, and similar facilities. There is also a large amount of statistical information on port matters, both maritime and economic.

TIDAL POWER.

By A. M. A. Struben. Lond. and N. Y., Sir Isaac Pitman & Sons, Ltd., 1921. 115 pp., diagrams, 6 x 4 in., boards. 85 cents.

This little book is intended to stimulate interest in a field that is likely to attract attention in the near future. It indicates some of the possibilities, the difficulties that are found, and the systems that have been proposed.

DONATIONS TO THE READING ROOM

A MANUAL OF THE PRINCIPAL INSTRUMENTS

Used in American Engineering and Surveying. Manufactured by W. & L. E. Gurley. Forty-Eighth Edition. Troy, N. Y., W. & L. E. Gurley, 1921. 333 pp., 6½ x 4½ in., cloth. $1.00.

In presenting this edition of their Manual, the publishers state that it is primarily a book of instruction in the adjustment and use of field instruments. Simplicity of expression has been sought, and no attempt has been made at treatises which are more properly to be found in technical publications. They offer the Engineering Profession many new field instruments, new features, and refinements, resulting from continuous effort and study for a period of 75 years.

L'EFFORT DU RESEAU DU NORD,

Pendant et après la Guerre. Par M. Javary. Lille, L. Danel, 1921. 124 pp., 9½ x 6½ in., paper. (Presented by William Barclay Parsons, M. Am. Soc. C. E.)

This book gives an account of the work done by the Northern Railways of France, during and after the World War, and their plans for the future. It was written on the presentation of the Gold Medal of the Société Industrielle du Nord de la France, to the Compagnie du Nord, in appreciation of work accomplished in the rehabilitation of the devastated regions.

THE BUILDING ESTIMATOR'S REFERENCE BOOK.

By Frank R. Walker, Affiliate Am. Soc. C. E. Fourth edition. Chic., Frank R. Walker Co., 1921. 2 931 pp., illus., diagrams, 6⅝ x 4½ in., leather. $10.00.

In the sub-title of the book, the author states that it is a practical and thoroughly reliable reference book for contractors and estimators, engaged in estimating the cost of and constructing all classes of modern buildings, giving the labor costs and methods used in the erection of some present-day structures, together with all the necessary material and labor quantities entering into the cost of all classes of buildings.

HANDBOOK DESCRIBING BERLOY BUILDING MATERIALS,

Compiled by Building Materials Division, Engineering Department of the Berger Manufacturing Co. Canton, Ohio, Berger Manufacturing Co., copyright 1921. 404 pp. $6\frac{5}{8}$ x $4\frac{1}{2}$ in., cloth. $2.00.

This book is intended for the use of architects and engineers in designing and detailing of metal lumber joist and stud construction, as well as reinforced concrete, and other fire-resistive structures, involving the use of Berloy pressed steel structural sections, floor-cores, rib-plex, lath, centering, and reinforcing plates.

DOCK AND LOCK MACHINERY:

A Technical Manual. By W. Henry Hunter, M. Am. Soc. C. E. Lond., Constable & Co., Ltd., 1921. 207 pp., diagrams, plates, $8\frac{5}{8}$ x $5\frac{1}{2}$ in., cloth. (Gift of James A. Orrell, Assoc. M. Am. Soc. C. E.)

In these pages, the author has traced the evolution of the various kinds of mechanical devices, which have been and are being provided to facilitate the transport of floating craft of any type and size from one waterway to another, including the gates, sluices, gear, etc., to their modern forms. The book contains, in a collected form, information which is otherwise scattered, and not easily accessible, and an endeavor has been made to insure accuracy of description, as well as of historical statement. Where opinions have been expressed, they have been based not only on precedent, but on personal experience.

MEMBERSHIP

(From November 2d to December 6th, 1921)

ADDITIONS

MEMBERS

		Date of Membership.
ADAMS, EDWIN LEARNED. Gen. Supt., Pipe Line Co. and General Petroleum Corporation, 1003 Higgins Bldg., Los Angeles, Cal.	Assoc. M. M.	Aug. 31, 1915 Nov. 21, 1921
BALDWIN, FRANCIS NEAL. Bridge and Bldg. Supervisor, Tex. & Pac. Ry., Denton, Tex.	Assoc. M. M.	April 1, 1914 Nov. 21, 1921
BROWN, ROBERT. Supt. of Constr., Magnolia Petroleum Co., Dallas, Tex.		Nov. 21, 1921
BUMP, ARCHIE EDMUND. 41 Melville Ave., Dorchester 24, Mass.		Sept. 12, 1921
BUTLER, WILLIAM PARKER. Cons. Engr., Southern Surety Co., 406 Independent Life Bldg., Nashville, Tenn.	Assoc. M. M.	Nov. 27, 1917 Nov. 21, 1921
CARBERRY, RAY SHEPPARD. Supt., Imperial Water Co., No. 1, Imperial, Cal.	Assoc. M. M.	Mar. 4, 1908 Nov. 21, 1921
CRANFORD, FREDERICK LOUD. Pres. and Treas., Frederick L. Cranford, 149 Remsen St., Brooklyn, N. Y.		Nov. 21, 1921
DANA, ALLSTON. Asst. Engr. of Design, Delaware River Bridge Joint Comm., 818 Widener Bldg., Philadelphia, Pa.		Nov. 21, 1921
ENGER, MELVIN LORENIUS. Prof., Mechanics and Hydraulics, Univ. of Illinois, 204 Eng. Hall, Urbana, Ill.		Nov. 21, 1921
FAUCETTE, WILLIAM DOLLISON. Chf. Engr., Seaboard A. L. Ry., 1228 Royster Bldg., Norfolk, Va.	Jun. Assoc. M. M.	Jan. 6, 1903 May 7, 1913 Oct. 12, 1921
GARMAN, HARRY OTTO. Chf. Engr., Public Service Comm. of Indiana, 2062 North Meridian St., Indianapolis, Ind.	Jun. Assoc. M. M.	Feb. 28, 1905 Oct. 7, 1908 Nov. 21, 1921
GRUNSKY, EUGENE LUCIUS. Cons. Engr. (C. E. Grunsky Co.), 57 Post St., San Francisco, Cal.	Assoc. M. M.	Nov. 3, 1915 Nov. 21, 1921
HUNTER, HARRY GRIFFITH. Asst. Engr., Harrington, Howard & Ash, 1012 Baltimore Ave. (Res., 101 Rock Spring Rd.), Kansas City, Mo.	Assoc. M. M.	May 31, 1916 Nov. 21, 1921
JOHNSTON, HARRY VESTER. Cons. Engr., 879 Monadnock Bldg., San Francisco, Cal.		Sept. 12, 1921
LANG, PHILIP GEORGE, JR. Engr. of Bridges, B. & O. R. R., 1300 B. & O. Bldg., Baltimore, Md.	Assoc. M. M.	Oct. 3, 1911 Nov. 21, 1921
MCELROY, JAMES ALOYSIUS. City Engr., City Hall, Bridgeport, Conn.		Nov. 21, 1921
MCENTEER, FRANK DUFF. Pres. and Gen. Mgr., Concrete Steel Bridge Co., Clarksburg, W. Va.		Nov. 21, 1921
MYERS, ERNEST LINDLEY. Cons. Engr. (Myers & Noyes), 311 Deere Bldg., Dallas, Tex.	Assoc. M. M.	Nov. 28, 1916 Nov. 21, 1921
MYERS, WILLIAM KURTZ. Valuation Mgr., Philadelphia Rapid Transit Co. and International Ry., 820 Dauphin St. (Res., 210 East Highland Ave., Chestnut Hill), Philadelphia, Pa.		Nov. 21, 1921
PARKER, JAMES LAFAYETTE. Bridge Engr., State Highway Dept., Columbia, S. C.	Jun. Assoc. M. M.	April 2, 1907 May 3, 1910 Nov. 21, 1921

MEMBERS (*Continued*)

Name and Address	Grade	Date of Membership
PECK, LEON FRIEND. Supt., Dept. of Streets, Municipal Bldg., Hartford, Conn	Assoc. M. M.	Sept. 2, 1914 Nov. 21, 1921
PINNER, GUY. Care, American Consulate, Cartegena, Colombia.	Assoc. M. M.	Nov. 12, 1913 Oct. 12, 1921
PRESTON, GEORGE HENRY. Acting Engr., Turner Constr. Co., 244 Madison Ave., New York City (Res., 131 Belleville Ave., Bloomfield, N. J.)	Assoc. M. M.	April 4, 1911 Nov. 21, 1921
RICE, GUY WICKLIFFE. Asst. Gen. Mgr. and Chf. Engr., California Southern R. R., Blythe, Cal.	Assoc. M. M.	Nov. 3, 1910 Nov. 21, 1921
SCHAEFER, CHARLES HENRY. Contr. Engr., U. S. Structural Co., Inc., 841 Broadway, New York City (Res., 4824 Marvine St., Philadelphia, Pa.)		Nov. 21, 1921
SHERIDAN, LAWRENCE VINNEDGE. Executive Secy., City Planning Comm., Indianapolis, Ind.	Jun. Assoc. M. M.	Mar. 5, 1912 Dec. 6, 1915 Nov. 21, 1921
STEESE, JAMÉS GORDON. Pres., Alaska Road Comm., Juneau, Alaska	Jun. Assoc. M. M.	Aug. 31, 1909 Dec. 3, 1913 Sept. 12, 1921
SWEENY, FRANCIS RAYMOND IZLAR. Cons. and Const. Engr. (Sanders & Sweeny), Box 18, Anderson, S. C		Nov. 21, 1921
WEBSTER, MAURICE ANDERSON. With Wm. Steele & Sons Co., 243 Winona Ave., Germantown, Philadelphia, Pa.	Jun. Assoc. M. M.	Sept. 2, 1914 May 15, 1917 Nov. 21, 1921
WILLIAMS, JACOB PAUL JONES. 86 Elm St., Flushing, N. Y.	Assoc. M. M.	July 1, 1909 Nov. 21, 1921
WILLIAMSON, JOHN FURROW. 614 West Boone St., Piqua, Ohio		June 6, 1921
YOUNG, STELL KAY. Secy. and Chf. Engr., Upper Scioto Conservancy Dist., Kenton, Ohio	Assoc. M. M.	Nov. 25, 1919 Nov. 21, 1921

ASSOCIATE MEMBERS

Name and Address	Grade	Date of Membership
ADAMS, THOMAS PATTON. Asst. Supt. of Constr., Stone & Webster, Inc., 1701 F. & M. National Bank Bldg., Fort Worth, Tex.		Nov. 21, 1921
ARNOLD, FRANK PALMER. Asst. Engr., West Virginia State Road Comm., 43 California Pl., Charleston, W. Va.	Jun. Assoc. M.	Sept. 9, 1919 Nov. 21, 1921
BALLARD, WILSON TURNER. Asst. Civ. Engr., Paving Comm., City of Baltimore, 1622 Mount Royal Ave., Baltimore, Md.		Nov. 21, 1921
BERNS, MAX ARNOLD. Publicity Mgr., Universal Portland Cement Co., 210 South La Salle St., Chicago, Ill.		Nov. 21, 1921
BROCKMEYER, EDWIN JOHN. Engr., J. P. Jamieson, 800 Security Bldg., St. Louis, Mo.		Oct. 10, 1921
BURTNER, GEORGE ROBERT. Asst. County Engr., 2003 Walworth St., Greenville, Tex.		Oct. 10, 1921
CAMPBELL, HENRY BOWERS. Hydrographic and Geodetic Engr., U. S. Coast and Geodetic Survey, Parishville, N. Y.		Oct. 10, 1921
CARMAN, ARTHUR ADAM AUGUSTINE. Cons. Engr., 280 Madison Ave., New York City (Res., 2107 Bedford Ave., Brooklyn, N. Y.)		Nov. 21, 1921
CECIL, NEIL MCCOMAS. Care, Don Pedro Dam, Turlock, Cal.		Oct. 10, 1921

ASSOCIATE MEMBERS (*Continued*)

		Date of Membership.
CONSTANT, CLYDE STANLEY. Supt. and Engr., The Kaw Paving Co., 616 New England Bldg., Topeka (Res., 743 Rhode Island St., Lawrence), Kans.	Jun. Assoc. M.	Oct. 14, 1919 Nov. 21, 1921
COOK, ARTHUR T. Chf. Engr., Santiago Min. Co., Casilla 83-D, Santiago, Chile.		Sept. 12, 1921
COOK, HOLTON. Res. Engr., Dept., State Roads and Highways of Kentucky, Smithland, Ky.	Jun. Assoc. M.	Aug. 31, 1915 Oct. 10, 1921
COTTON, HAROLD ALONZO. Care, Director of Coast Surveys, U. S. Coast and Geodetic Survey, Manila, Philippine Islands		June 6, 1921
DAVIS, ROBERT WAITE. Res. Engr., State Highway Comm., Slidell, La.		Nov. 21, 1921
DAVIS, THOMAS MARSH. Highway Engr., U. S. Bureau of Public Roads, 246 East 53d St., Portland, Ore.		Nov. 21, 1921
DOERR, CHARLES WILLIAM. Asst. Engr., Erecting Dept., Am. Bridge Co., Frick Bldg., Pittsburgh (Res., 28 Terrace Ave., Emsworth, Bellevue P. O.) Pa.	Jun. Assoc. M.	Aug. 31, 1915 Oct. 10, 1921
EVANS, CHARLES DORMAN. Cons. Engr., 311 Levy Bldg., Shreveport, La.		Mar. 7, 1921
EWIN, JAMES PERKINS. Chf. Engr., Doullut & Williams Co., Inc., 816 Howard Ave., New Orleans, La.	Jun. Assoc. M.	April 16, 1918 Nov. 21, 1921
FICKES, EUGENE WELDON. Asst. Engr., John H. Wickersham, 110 Ruby St., Lancaster, Pa.	Jun. Assoc. M.	Sept. 11, 1917 Sept. 12, 1921
FOSTER, HOWARD LESLIE. Asst. Engr., Lockwood, Greene & Co., 4073 Pingree Ave., Detroit, Mich.	Jun. Assoc. M.	April 19, 1920 Nov. 21, 1921
FROST, HARRY EDWIN. 30 Buckmenster St., Suite 9, Allston, Mass.		Sept. 12, 1921
FUGITT, LEROY BURROWS. Asst. Engr. of Constr., Kansas City Bridge Co., 510 Orear-Leslie Bldg., Kansas City, Mo.		Nov. 21, 1921
GALBRAITH, ALAN LOVE. Civ. Engr., State Electricity Comm. of Victoria, 30 William St., Melbourne, Victoria, Australia		June 6, 1921
HARGETT, FREDERICK LOUIS. Supt. of Constr., J. A. Burt, Mineral Springs, Ark.		Nov. 21, 1921
HIET, MELVIN EMERSON. County Engr., Pawnee and Nowata Counties, Box 791, Nowata, Okla.		Nov. 21, 1921
HOLTMAN, DUDLEY FRANK. Constr. Engr., National Lumber Mfrs. Assoc., 721 Southern Bldg., Washington, D. C.		Nov. 21, 1921
HOOPER, ELMER GUY. Asst. Prof., Civ. Eng., New York Univ., 181st St. and University Ave., New York City		Nov. 21, 1921
JOHNSON, ALGOT FERDINAND. 809 First National Soo Line Bldg., Minneapolis, Minn.		Nov. 21, 1921
KEAST, SCHUYLER SHELDON ALBERT. Asst. Engr., Dept. of City Transit (Res., 1615 Mentor St., Logan), Philadelphia, Pa.		Nov. 21, 1921
KELLAM, FRED. Testing Engr., Indiana State Highway Comm., 510 West Market St., Indianapolis, Ind.		Oct. 10, 1921
KELLY, JAMES MICHAEL. 2017 South 66th St., Philadelphia, Pa.		Nov. 21, 1921
LANE, HERRICK JOHNSON. Office Engr., Doullut & Williams Co., Inc., 816 Howard Ave., New Orleans, La.		Nov. 21, 1921
LANCET, KENNETH EARL. 2358 Broadway, Indianapolis, Ind.		Nov. 21, 1921
LIVESAY, WALLACE BRIGHT. Chf. Draftsman, Port Neches Plant of The Texas Co., Port Neches, Tex.		Nov. 21, 1921

ASSOCIATE MEMBERS (*Continued*)

	Date of Membership.
LOVELL, CHARLES WILLIAM. Res. Engr., Dept., State Roads and Highways, 968 South 2d St., Louisville, Ky.	Nov. 21, 1921
MCBRIDE, BERNARD REEVES. 917 East 5th St., Columbus, Ind.	Nov. 21, 1921
MCCOY, JOSEPH MUTH. Sales and Field Engr., Redwood Mfrs. Co., 1600 Hobart Bldg., San Francisco (Res., Red Bluff), Cal.	Nov. 21, 1921
MARION, JOHN MICHAEL. Care, A. E. Alexander, 2031 Eastern Parkway, Brooklyn, N. Y.	June 6, 1921
MATTER, LESTER DONALD. 507 Moore St., Henryetta, Okla.	June 6, 1921
MELLISH, MURRAY HOLMAN. 174 Ferry St., Malden, Mass.	Nov. 21, 1921
MEYER, HENRY RUPERT JOHN. City Engr., Box 144, Havre, Mont.	July 11, 1921
MULDROW, WILLIAM CANON. With Cascade Constr. Co., 405 Douglas Bldg., Seattle, Wash.	Sept. 12, 1921
NICHOLLS, CARROLL CLIFFORD. Res. Engr., Grant, Fulton & Leeton, 505 Bankers Life Bldg., Lincoln (Res., 2009 J St., University Place), Nebr.	Oct. 10, 1921
NISENSON, AMOS OSCAR. Civ. Engr. and Surv. (Lake & Nisenson), 9 Clinton St., Newark, N. J.	June 6, 1921
NORD, CHARLES LOUIS. 716 Dixwell Ave., New Haven, Conn.	Jan. 17, 1921
PORTER, CHARLES ROBERT. Care, The Engr., Tees Conservancy Commrs., Middlesbrough, England	Oct. 10, 1921
PORTER, HENRY CYRUS. County Highway Engr., Kleberg County, Kingsville, Tex.	Nov. 21, 1921
PRAEGER, EMIL. Structural Engr., B. G. Goodhue, 52 Stratford Rd., Brooklyn, N. Y. { Jun. / Assoc. M.	Dec. 6, 1915 / Nov. 21, 1921
RAFTER, CASE BRODERICK. Structural Engr., 1002 Hibbs Bldg., Washington, D. C.	Oct. 10, 1921
RAYMOND, LAWRENCE ELMER. Supt. of Constr. with W. L. Stoddart, Box 94, High Point, N. C.	Nov. 21, 1921
REMSEN, PETER. 802 Nineteenth St., N. W., Washington, D. C.	Nov. 21, 1921
RODGERS, WILLIAM EVANS. Contractor's Engr., for J. H. Cahill, 643 South 8th St., Louisville, Ky.	Nov. 21, 1921
ROSENTHAL, CERF. Cons. Engr. (Rosenthal & Davis), 1315 Humboldt Bank Bldg., San Francisco, Cal.	Nov. 21, 1921
SARVIS, FRED WHITE. County Engr., Shelby County, Harlan, Iowa	Oct. 10, 1921
SHRIVER, RAY OTTO. Municipal Contr. (C. H. Everett Co.), 201 W. S. 5th St., Newton, Kans.	Nov. 21, 1921
STILL, JOSEPH FRANCIS. Highway Engr., Bureau of Public Roads, Dist. No. 8, U. S. Dept. of Agriculture, St. Petersburg, Fla.	Nov. 21, 1921
TALLMADGE, ALVAN BRASEE. Asst. Mgr., J. J. Morgan Co., 301 Gugle Bldg., Columbus, Ohio	June 6, 1921
THOMPSON, RALPH PENNY. Masonry Insp., Interstate Ry., Box B, Coeburn, Va.	Nov. 21, 1921
TODD, EDWARD NEWTON. 1306 West 23d St., Oklahoma, Okla.	Nov. 21, 1921
WEBER, ADOLPH GOTTIG. 2618 Cedar St., Berkeley, Cal.	Oct. 10, 1921
WIEGNER, CHAUNCEY J. Chf. Engr. of Location, Owl Drainage Dist. of Scotland and Clark Counties, Memphis, Mo.	Nov. 21, 1921

AFFILIATES

WINELL, VERN ELWOOD. Care, Thompson-Starrett Co., 51 Wall St., New York City	Nov. 21, 1921

JUNIORS	Date of Membership.
BREUER, WILLIAM, 2736 West Montgomery Ave., Philadelphia, Pa.	Nov. 21, 1921
BROWN, REX LENOI. 300 Laboratory of Applied Mechanics, Univ. of Illinois, Urbana (Res., 809 South 1st St., Champaign), Ill.	Oct. 10, 1921
DYER, HARRY BUTTORFF. Draftsman, Nashville Bridge Co., 2503 Kensington Pl., Nashville, Tenn.	Nov. 21, 1921
EASTMAN, EDMUND MADISON. 239 Oak St., Atlanta, Ga.	Nov. 21, 1921
FLITTNER, FRANK WILLIAM. Eng. Dept., Standard Oil Co., 200 Bush St., San Francisco, Cal.	Nov. 21, 1921
GORMAN, SIDNEY SILVEY. Asst. Engr., J. B. Leonard, 5 State St., San Francisco, Cal.	Sept. 12, 1921
GOUDEY, RAY FREEMAN. Asst. Engr., Bureau of San. Eng., California State Board of Health, 821 Pacific Finance Bldg., Los Angeles, Cal.	Nov. 21, 1921
HANSEN, REINHOLD BERNHARD. Office Engr., Standard Oil Co., 999 Dolores St., San Francisco, Cal.	Oct. 10, 1921
LEVIN, ABRAHAM. Draftsman, Buckingham Steel Co., 679 Second Ave., New York City.	Nov. 21, 1921
McGRATH, WILLIAM JOHN. 460 West 150th St., New York City.	Oct. 10, 1921
SPEAR, CARLTON JERNEGAN. Supt. of Constr., Roger Black Co., 452 Lexington Ave. (Res., Edgartown, Mass.), New York City.	Nov. 21, 1921
STORMS, HAROLD BEEKMAN. 350 South Third Ave., Mount Vernon, N. Y.	Nov. 21, 1921
WARNER, FAYETTE SAMUEL. Sunderland, Mass.	Oct. 10, 1921
WILBUR, LYMAN DWIGHT. 241 Frederick St., San Francisco, Cal.	Oct. 10, 1921

REINSTATEMENTS

MEMBERS	Date of Reinstatement.
HAYNES, GEORGE ALBERT	Oct. 11, 1921
ASSOCIATE MEMBERS	
SAMPLE, WILLIAM DWIGHT	Nov. 9, 1920

RESIGNATIONS

MEMBERS	Date of Resignation.
RICHARDSON, CLIFFORD	Oct. 10, 1921
ASSOCIATE MEMBERS	
BUCHANAN, NATHAN BOOKER	Dec. 31, 1920

DEATHS

BAKER, WILLIAM EDGAR. Elected Member, June 1st, 1898; died November 7th, 1921.

BATTLE, JOHN BEALIE. Elected Member, October 9th, 1917; died November 6th, 1921.

BRADLEY, CHARLES WHITING. Elected Affiliate, June 19th, 1921; died January 14th, 1920.

BROWNE, JAMES SIMPSON. Elected Associate Member, October 4th, 1893; Member July 1st, 1909; died October 22d, 1921.

Burrows, George Lord. Elected Affiliate, February 3d, 1886; died November 9th, 1921.

Chipley, Dudley. Elected Associate Member, September 11th, 1917; died August 20th, 1921.

Fox, *Sir* Douglas. Elected Corresponding Member, June 7th, 1871; Honorary Member, March 5th, 1901; died November 13th, 1921.

Goodwyn, Richard Tuggle, Jr. Elected Associate Member, September 12th, 1921; died November 8th, 1921.

Gray, Samuel Merrill. Elected Member May 15th, 1872; died November 6th, 1921.

Hains, Peter Conover. Elected Member, April 2d, 1890; died November 7th, 1921.

Harvie, Henry. Elected Associate Member, May 12th, 1919; died October 14th, 1921.

Holmes, Howard Carleton. Elected Member, November 4th, 1903; died October 30th, 1921.

Perkins, Philo Sackett. Elected Junior, March 5th, 1890; Associate Member, April 3d, 1895; Member, July 11, 1921; died October 28th, 1921.

Quintus, John Charles. Elected Member, January 2d, 1889; died November 27th, 1921.

Raymond, Charles Ward. Elected Junior, November 7th, 1877; Member, April 7th, 1886; died October 27th, 1921.

Robson, Ralph Ewart. Elected Junior, October 3d, 1911; Associate Member, June 3d, 1915; died October 14th, 1921.

Scheuermann, Hugo Julius. Elected Associate Member, December 7th, 1904; died July 20th, 1921.

Webb, George Herbert. Elected Member, February 1st, 1893; died November 3d, 1921.

Whistler, Thomas Delano. Elected Junior, March 5th, 1884; Member May 2d, 1888; date of death unknown.

Wrenn, James Francis. Elected Affiliate, September 6th, 1905; died November 2d, 1921.

Total Membership of the Society, December 6th, 1921, 10 333.

MONTHLY LIST OF RECENT ENGINEERING ARTICLES OF INTEREST

(October 29th to December 1st, 1921)

NOTE.—*This list is published for the purpose of placing before the members of this Society the titles of current engineering articles, which can be referred to in any available engineering library, or can be procured by addressing the publication directly, the address and price being given wherever possible.*

LIST OF PUBLICATIONS

In the subjoined list of articles, references are given by the number prefixed to each journal in this list.

(2) *Journal,* Engrs. Club of Phila., Philadelphia, Pa.
(3) *Journal,* Franklin Inst., Philadelphia, Pa., 50c.
(4) *Journal,* Western Soc. of Engrs., Chicago, Ill., 50c.
(5) *Journal,* Eng. Inst. of Canada, Montreal, Que., Canada.
(6) *Journal,* Am. Inst. of Archts., Washington, D. C., 50c.
(7) *Gesundheits Ingenieur,* Munich, Germany.
(8) *Stevens Indicator,* Hoboken, N. J., 50c.
(9) *Industrial Management,* New York City, 25c.
(11) *Engineering* (London), W. H. Wiley, 432 Fourth Ave., New York City, 25c.
(12) *The Engineer* (London), International News Co., New York City, 35c.
(13) *Engineering News-Record,* New York City, 25c.
(15) *Railway Age,* New York City, 15c.
(16) *Engineering and Mining Journal,* New York City, 15c.
(17) *Electric Railway Journal,* New York City, 10c.
(18) *Railway Review,* Chicago, Ill., 15c.
(20) *Iron Age,* New York City, 20c.
(21) *Railway Engineer,* London, England, 1s 2d.
(22) *Iron and Coal Trades Review,* London, England, 6d.
(24) *American Gas Journal,* New York City, 10c.
(25) *Railway Mechanical Engineer,* New York City, 20c.
(26) *Electrical Review,* London, England, 4d.
(27) *Electrical World,* New York City, 10c.
(28) *Journal,* New England Water-Works Assoc., Boston, Mass., $1.
(29) *Journal,* Royal Soc. of Arts, London, England, 6d.
(30) *Annales des Travaux Publics de Belgique,* Brussels, Belgium.
(31) *Annales* de l'Assoc. des Ingenieurs Sortis des Ecoles Speciales de Gand. Brussels, Belgium.
(32) *Memoirs et Compte Rendu des Travaux,* Soc. Ing. Civ. de France, Paris, France.
(33) *Le Génie Civil,* Paris, France, 1 fr.
(36) *Cornell Civil Engineer,* Ithaca, N. Y.
(40) *Zentralblatt der Bauverwaltung,* Berlin, Germany, 60 pfd.
(41) *Elektrotechnische Zeitschrift,* Berlin, Germany.
(42) *Journal,* Am. Inst. Elec. Engrs., New York City, $1.
(43) *Annales des Ponts et Chausées,* Paris, France.
(45) *Coal Age,* New York City, 15c.
(46) *Scientific American,* New York City, 35c.
(47) *Mechanical Engineer,* Manchester, England, 3d.
(48) *Zeitschrift,* Verein DeutscherIngenieure, Berlin, Germany.
(49) *Zeitschrift für Bauwesen,* Berlin, Germany.
(50) *Stahl und Eisen,* Düsseldorf, Germany.
(53) *Zeitschrift,* Oesterreichischer, Ingenieur und Architekten-Verein, Vienna, Austria, 70h.
(54) *Transactions,* Am. Soc. C. E., New York City, $16.
(55) *Mechanical Engineering:* Journal, Am. Soc. M. E., New York City, 35c.
(57) *Colliery Guardian,* London, England, 5d.
(58) *Proceedings,* Engrs.' Soc. of W. Pa., 2511 Oliver Bldg., Pittsburgh, Pa., 50c.
(59) *Proceedings,* American Water Works Assoc., Troy, N. Y.
(60) *Municipal and County Engineering,* Indianapolis, Ind., 25c.
(61) *Proceedings,* Western Railway Club, 225 Dearborn St., Chicago, Ill., 25c.
(62) *Forging and Heat Treating,* Thaw Bldg., Pittsburgh, Pa., 10c.
(63) *Minutes of Proceedings,* Inst. C. E., London, England.
(64) *Power,* New York City, 15c.
(65) *Official Proceedings,* New York Railroad Club, Brooklyn, N. Y., 15c.
(67) *Cement and Engineering News,* Chicago, Ill., 25c.
(69) *Eisenbau,* Leipzig, Germany.
(71) *Journal,* Iron and Steel Inst., London, England.
(71*a*) *Carnegie Scholarship Memoirs,* Iron and Steel Inst., London, England.
(72) *American Machinist,* New York City, 15c.
(73) *Electrician,* London, England, 1s.
(75) *Proceedings,* Inst. of Mech. Engrs., London, England.
(77) *Journal,* Inst. Elec. Engrs., London, England, 5s.
(78) *Beton und Eisen,* Vienna, Austria.
(80) *Tonindustrie Zeitung,* Berlin, Germany.
(83) *Gas Age-Record,* New York City, 15c.
(85) *Proceedings,* Am. Ry. Eng. Assoc., Chicago, Ill.
(86) *Engineering and Contracting,* Chicago, Ill., 10c.

(87) *Railway Maintenance Engineer,* Chicago, Ill., 10c.
(88) *Bulletin* of the International Ry. Congress Assoc., Brussels, Belgium.
(89) *Proceedings,* Am. Soc. for Testing Materials, Philadelphia, Pa., $5.
(90) *Transactions,* Inst. of Naval Archts., London, England.
(91) *Transactions,* Soc. of Naval Archts. and Marine Engrs., New York City.
(92) *Bulletin,* Soc. d'Encouragement pour l'Industrie Nationale, Paris, France.
(96) *Canadian Engineer,* Toronto, Ont., Canada, 10c.
(98) *Journal,* Engrs. Soc. of Pa., Harrisburg, Pa., 30c.
(99) *Proceedings,* Am. Soc. of Municipal Improvements, New York City, $2.
(100) *Military Engineer:* Journal of the Society of American Military Engineers, Washington, D. C., 75c.
(103) *Mining and Scientific Press,* San Francisco, Cal., 10c.
(105) *Chemical and Metallurigical Engineering,* New York City, 25c.
(106) *Transactions,* Inst. of Min. Engrs., London, England, 6s.
(107) *Schweizerische Bauzeitung,* Zürich, Switzerland.
(109) *Journal,* Boston Soc. C. E., Boston, Mass., 50c.
(111) *Journal of Electricity,* San Francisco, Cal., 25c.
(113) *Proceedings,* Am. Wood Preservers' Assoc., Baltimore, Md.
(114) *Journal,* Institution of Municipal and County Engineers, London, England, 1s. 6d.
(115) *Journal,* Engrs. Club of St. Louis, St. Louis, Mo., 35c.
(116) *Blast Furnace and Steel Plant,* Pittsburgh, Pa., 15c.
(117) *Engineering World,* Chicago, Ill.
(118) *Times Engineering Supplement,* London, England, 2d.
(119) *Landscape Architecture,* Harrisburg, Pa., 50c.
(120) *Automotive Industries,* New York City, 15c.
(121) *Proceedings,* Am. Concrete Inst., Boston, Mass.
(122) *The Dock and Harbour Authority,* London, England, 1s. 6d.
(123) *Mining and Metallurgy,* New York City, $1.

LIST OF ARTICLES

Bridges.

Crossing San Francisco Bay by Bridge, Fill and Tunnel.* (46) Nov.
Some Thoughts on Long-Span Bridge Design. Gustave Lindenthal. (13) Nov. 24.
Calcul des Ponts Circulaires à une Suele Travée et à Travées Continues.* (Calculation of Curved Bridges with Single and Continuous Trusses.) M. Bertrand de Fontviolant. (32) July-Sept., 1920.
Note sur l'Emploi des Palplanches Métalliques dans les Travaux de Reconstructión des Ponts sur la Meuse détruits pendant la Guerre de 1914–1918.* (Note on the Use of Metallic Sheet Piling in the Work of Restoration of the Bridges Over the Meuse Destroyed During the War from 1914 to 1918.) M. Wisdorff et M. Chopinet. (38) Oct.
Neuartige Klappbrücke in der Strassenbrücke über die Eiderbei Friedrichstadt.* (New Kind of Bascule Bridge for the Highway Bridge Over the Eider at Friedrichstadt.) (40) July 31, 1920.
Regelquerschnitte für Strassenbrücken.* (Standard Cross-Sections for Street Bridges.) Ellerbeck and Starker. (40) July 28, 1920.
Der Umbau der Eisenbahnkriegsbrücke über die Memel bei Olita.* (The Rebuilding of the Military Railway Bridge Over the Memel at Olita.) (40) Aug. 7, 1920.
Der Umbau der Landpfeiler der Stadtbahnbrücke über die Spree am Bahnhof Bellevue in Berlin.* (Rebuilding the Shore Piers of the Municipal Railway Bridge Over the Spree to the Bellevue Station in Berlin.) Kuhnke. (40) Sept. 11, 1920.
Wiederherstellung der zerstörten Eisenbahnbrücke über die Weichsel bei Warschau in der Linie nach Wilna.* (Restoration of the Destroyed Railway Bridge Over the Weichsel at Warsaw in the Line to Vilna.) (40) Oct. 20, 1920.
Der Abbau der gesprengten Brücke über die Memel bei Grodno.* (Demolishing the Bridge Over the Memel at Grodno That Was Blown Up.) (40) Oct. 30, 1920.
Die zweigleisige Johannestahlbrücke im Zuge des Eisenbahnstrecke Hannover-Hamm.* (The Johannes Double Track Steel Bridge in the Hannover-Hamm Railway Division.) Gaede. (40) Nov. 13, 1920.
Die Eisenbahnbrücke uber das Hollen-thor in Neuyork.* (The Railway Bridge Over Hell Gate at New York.) Schinkel. (40) Dec. 18, 1920.
Eiserne Brücken in Stadtbilde. (Iron Bridges Within the City.) Karl Bernhard. (48) Oct. 15.
Ueber die Bewegungen der Hauptpfeiler-Köpfe der Trisannabrücke an der Arlbergbahn.* (On the Movements of the Top of the Main Pier of the Trisanna Bridge on the Arlberg Railway.) Leopold Orley. (107) Oct. 29.

Electrical.

Development of Army Wireless During the War. A. G. T. Cusins. (77) July.
Abnormal Pressure-Rise in Transformers, and Its Remedy.* R. Torikai. (77) July.
Electric Oscillations in Straight Wires and Solenoids.* J. S. Townsend. (77) July.
Electric Supply: Present Conditions and the Hopkinson Principles.* J. R. Blaikie. (77) July.
Multi-Part Tariffs for Domestic Electricity Supply.* J. W. Beauchamp. (77) July.
The Negatron: A New Negative Resistance Device for Use in Wireless Telegraphy.* John Scott-Taggart. (Paper read before British Assoc.) (11) Oct. 21.
Long Distance Transmission of Electrical Energy, With Special Reference to Tidal Power. T. F. Wall. (Paper read before British Assoc.) (11) Oct. 21.
Electricity in Isolated Buildings.* E. H. Freeman. (26) Serial beginning Oct. 28.

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* Illustrated.

Electrical—(Continued).

Practical Method for Calculating Operating Characteristics of Magnet Coils.* Charles R. Underhill. (27) Oct. 29.
Revision of Some of the Electro-magnetic Laws.* Carl Hering. (3) Nov.
Electric Propulsion of Ships.* W. E. Thau. (42) Nov.
A Frequency-Bridge.* Edy Velander. (42) Nov.
Abnormal Voltage on Y-Delta Transformer Bank Due to Reversed Connection.* C. R. Reid. (42) Nov.
Commutation on Direct-Current Machines.* Claudius Sheufer. (42) Nov.
Physical Conceptions of Induction Motor Operation.* J. Lebovicé. (42) Nov.
Use of the Tangent Chart for Solving Transmission Line Problems.* Raymond S. Brown. (42) Nov.
Recent Advances in the Production and Application of X-Rays.* J. S. Shearer. (3) Nov.
Measurements of Earth Currents.* Burton McCollum. (17) Nov. 5.
Improved Practices in Exterior and Interior Illumination.* (From Comm. Report, Illuminating Eng. Soc.) (27) Nov. 12.
Features of 220 000-Volt Transformers Available for First Time.* Walter M. Dann. (27) Nov. 26.
Les Troubles Provoqués par la Traction Electrique dans les Transmissions Télégraphiques et Téléphoniques.* (Troubles in Telegraph and Telephone Transmission Caused by Electric Traction.) J. Lhériaud. (32) Oct.-Dec., 1919.
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Marine.

Low-Pressure Turbine Blading Failures in Destroyers.* D. F. Ducey. (11) Serial beginning Oct. 28.
Electric Propulsion of Ships.* W. E. Thau. (42) Nov.
Les Sous-Marins Allemands.* (German Submarines.) M. Laubeuf. (32) Jan.–Mar., 1920.
La Direction et le Controle Automatiques de la Marche des Navires, à l'Aide d'Appareils Gyroscopiques Automatiques.* (Automatic Steering and Control of Vessels by Means of Automatic Gyroscopic Apparatus.) E. Weiss. (33) Oct. 8.
Chalands Métalliques de 700 Tonnes, Type H. Lossier, pour la Navigation sur la Seine.* (700 Ton Metallic Lighter, H. Lossier Type, for Navigation on the Seine.) Ch. Dantin. (33) Oct. 22.
L'Installation de Chargement du Charbon à bord des Navires à Curtis Bay (Maryland, E. U.).* (Installation for Loading Vessels With Coal at Curtis Bay (Maryland, U. S.).) (33) Oct. 22.
Das Verhalten des Eisens in See-wasser und Mittel zur Erhöhung der Lebensdauer eiserner Uferbauten.* (The Action of Iron in Seawater and Ways of Increasing the Life of Iron Coast Constructions.) Richard Ederhof. (40) Apr. 17, 1920.
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Mechanical.

New Electricity Generating Station at Blackburn.* (12) Oct. 21.
Coke as a Fuel for Commercial Vehicles.* Thomas Clarkson. (11) Oct. 21.
A Machine for the Measurement of Internal Diameters.* G. A. Tomlinson. (11) Oct. 21
Liberation of Nitrogen from Coal and Coke as Ammonia. A. C. Monkhouse and J. W. Cobb. (Paper read before Inst. of Gas Engrs.) (22) Oct. 21.
The Manufacture of Smokeless Fuel.* (11) Oct. 28.
Low Temperature Carbonization.* (57) Oct. 28.
Silica Brick for Coke Ovens.* A. H. Middleton. (Abstract of paper read before Coke Oven Managers' Assoc.) (57) Oct. 28.
Plant for Liquid Purification of Gas.* (83) Oct. 29.
Steam-Condensing Plants.* Paul A. Bancel. (55) Nov.
Control of Centrifugal Casting by Calculation.* Robert F. Wood. (55) Nov.
Fuel Saving in Relation to Capital Necessary.* Joseph Harrington. (55) Nov.
Boiler-Plant Efficiency.* Victor J. Azbe. (55) Nov.
Fuel Saving in Modern Gas Producers and Industrial Furnaces.* W. B. Chapman. (55) Nov.
The Science of Electric Welding.* W. E. Ruder. (3) Nov.
Experimental Work in Connection with the Manufacture of Silica Brick.* A. W. McMaster. (5) Nov.
Sterling, Colo., Manufacturing Concrete Sewer Pipes.* Glenn Izett. (117) Nov.
Burning Powdered Coal and Blast-Furnace Gas at River Rouge.* Thomas Wilson. (64) Nov. 1.
Copper Tube Extension and the Manufacture of Radiator Cores.* Herbert Chase. (120) Nov. 3.
Investigation of Tooth Wear With Automobile Gear Steels.* E. R. Ross. (120) Nov. 3.
Sintering Flue Dust With Minimum Labor.* H. V. Schiefer. (20) Nov. 3.
Making Dies for Forming Automobile Parts.* Richard Dale. (20) Nov. 3.
The Centrifugal Pump.* S. F. Barclay. (Abstract of paper read before British Assoc.) (12) Nov. 4.

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Economies in Coal Consumption at Collieries.* H. O. Dixon. (Paper read before Nat'l. Assoc. of Colliery Mgrs.) (22) Nov. 4.
Disposal of Waste from Gas Plants.* F. W. Sperr, Jr. (83) Nov. 5.
Apparatus for Scrubbing and Cooling. R. A. McNeas. (Paper read before South-Central Gas Assoc.) (83) Nov. 5.
Steam Power from Blast-Furnace Gas.* Gordon Fox and F. H. Willcox. (64) Nov. 8.
Enameled Steel Manufacture.* Chester H. Jones. (105) Nov. 9.
Stress Coefficients for Large Horizontal Pipes.* James M. Paris. (13) Nov. 10.
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Locating Electrical Trouble on Elevators.* William Zepernick. (64) Nov. 15.
Heating and Its Relation to Isolated-Plant Operation.* E. L. Wilder. (64) Nov. 15.
Investigation of Breakdown of 30 000-k. w. Turbine.* (64) Nov. 22.
The Several Efficiencies of the Steam Engine.* F. R. Low. (64) Nov. 22.
Ball Mills Pulverize Coal Almost Without Repair and Attendance Costs and With Minimum Power.* (45) Nov. 24.
Fusion Welding and the Processes in Use.* S. W. Miller. (Paper read before Am. Iron and Steel Inst.) (20) Nov. 24.
Burning Pulverized Anthracite Mine Waste.* O. M. Rau. (17) Nov. 26.
Carbonization at Low Temperature.* J. D. Davis. (83) Nov. 26.
Les Moteurs d'Aviation leur Evolution pendant la Grande Guerre.* (Aviation Engines: Their Development During the Great War.) Le Commandant Martinot-Lagarde. (32) Oct.-Dec., 1919.
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L'Utilisation Rationelle des Combustibles.* (Rational Utilization of Fuels.) M. Georges Charpy. (32) Apr.-June, 1920.
La Consommation de Charbon dans la Grosse Métallurgie.* (The Consumption of Coal in Large Scale Metallurgy.) M. E. de Loisy. (32) Apr.-June, 1920.
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L'Emploi du Charbon Pulvérisé aux Usines Sidérurgiques de la Knoxville Iron Company (Tennessee, E. U.).* (The Use of Pulverized Coal in the Iron Works of the Knoxville Iron Company (Tennessee, U. S.).) (33) Oct. 8.
Le Gazogène à Fusion des Cindres, Ses Applications, son Avenir.* (The Molten Ash Gas Producer. Its Applications and Future.) A. Fichet. (33) Oct. 15.
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Eine Förderanlage von hoher Wirtschaftlichkeit.* (A Conveying Installation of Great Efficiency.) G. Schlesinger. (48) Nov. 13, 1920.
Das allgemeine Verhalten der Kreiselverdichter.* (The Ordinary Action of the Centrifugal Compressor.) Gustav Flügel. (48) Dec. 4, 1920.
Aufzuganlage mit Fernsteuerung.* (Hoisting Installation with Remote Control.) (107) Feb. 26.
Versuche mit Zellstofftreibriemen.* (Experiments with Cellulose Belting.) Rudeloff. (48) Oct. 1.
Die Apparate für technische Gasanalyse.* (Apparatus for Technical Gas Analysis.) K. Aschof. (50) Oct. 6.
Die Beurteilung von Kaminkühlern.* (Criticism of Stack Coolers.) Kurt Neumann. (48) Oct. 8.
Die Behandlung der Werkzeuge in der Fabrik.* (The Treatment of Tools in the Factory.) A. Fattler. (48) Oct. 8.
Ersparnismöglichkeiten im Kokerei- und Nebengewinnungsbetriebe unter besonderer Berücksichtigung der Wärmewirtschaft.* (Possibilities for Economies in Coke Oven and By-Product Operation, With Special Consideration of Heat Economy.) W. Wollenweber. (50) Oct. 13.
Querschnittsübergänge und Biegefestigkeit bei Dauerbeanspruchung durch Stösse.* (Variation in Cross-Section and Bending Strength by Repeated Impact.) W. Müller and Hugo Leber. (48) Oct. 15.
Beitrag zur Zahnradfrage für Uebersetzungsgetriebe.* (Contribution to the Question of Toothed Wheels for Transmission Gearing.) O. Lasche. (48) Oct. 15.
Die Gleichungen des Verbrennungsvorganges.* (The Equations of the Process of Combustion.) R. Mollier. (48) Oct. 15.
Abgasanalytische Fluchtlinien-Rechentafeln zweiter Art (für kollektive Verbrennung).* (Two Kinds of Graphic Tables for Waste Gas Analysis (for Collective Combustion).) Wa. Ostwald. (50) Oct. 20.
Untersuchung einer schweren Senkrechtfräsmaschine.* (Investigation of a Heavy Vertical Milling Machine.) Willi Mitan. (48) Oct. 22.

Metallurgical.

Admiralty Gun-Metal.* R. T. Rolfe. (Abstract of paper read before Inst. of Metals.) (11) Oct. 21.
The Density of the Copper-Zinc Alloys. T. G. Bamford. (Abstract of paper read before Inst. of Metals.) (11) Oct. 21.

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Stainless Steel.* (26) Oct. 28.
Temperature Problems in Foundry and Melting Room. John P. Goheen. (123) Nov.
A Bibliography and Abstracts of Chromium Steels. F. P. Zimmerli. (105) Nov. 2.
Nitrogen in Carburized Steels.* W. E. Ruder and G. R. Brophy. (105) Nov. 9.
Bessemer Plant of Steel & Tube Company.* Gilbert L. Lacher. (20) Nov. 10.
Chloridizing Volatilization—Some Experiments and Their Practical Application.* Harai R. Layng. (16) Nov. 12.
National Tube Co.'s New Plant for Hammer-Welded Pipe.* Ernest Edgar Thum. (105) Nov. 16.
Impact Properties of Various Steels.* F. C. Langenberg. (105) Nov. 16.
Composition of Pig Iron and of Cast Iron. T. A. Dyer. (20) Nov. 17.
Concentrated HCl as Metallographic Etching Reagent for Nickel.* Henry S. Rawdon and Marjorie G. Lorentz. (105) Nov. 23.
Structure and Properties of Alternately Electro-Deposited Metals.* William Blum. (Paper read before Am. Electro-Chemical Soc.) (105) Nov. 23.
Improvements in Open-Hearth Port Construction. John W. Kagarise. (Paper read before Am. Iron and Steel Inst.) (20) Nov. 24.
Magnetic Analysis of Steel.* R. L. Sanford. (72) Nov. 24.
La Consommation de Charbon dans la Grosse Métallurgie.* (The Consumption of Coal in Large Scale Metallurgy.) M. E. De Loisy. (32) Apr.-June, 1920.
Le Nickelage de l'Aluminium.* (Nickel-Plating Aluminum.) M. Léon Guillet. (32) July-Sept., 1920.
Les Laitons Spéciaux.* (Special Brasses.) Leon Guillet. (32) Jan.-Mar., 1920.
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Die Scherprobe in ihrer Anwendung bei Gusseisen.* (The Shearing Test and Its Use with Cast Iron.) Karl Sipp. (50) Dec. 23-30, 1920.
Die Anlagen des Stahlwerkes Thyssen, A.-G., in Hagendingen (Lothr).* (The Equipment of the Thyssen Steel Works A.-G. in Hagendingen (Lothr).) F. Dahl. (50) Mar. 31.
Beiträge zur Frage der Wärmeformgebung schwerer Blöcke aus Schnellarbeitsstahl.* (Contribution to the Question of Hot Shaping of Heavy Blocks of High Speed Steel.) (50) Oct. 6.
Ueber ein neues Verfahren zur Bestimmung des Sauerstoffs im Eisen.* (On a New Method for the Determination of Oxygen in Iron.) O. von Keil. (50) Oct. 13.
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Das Vorkommen und Verhalten von Titan im Roheisenmischer. (The Existence and Behavior of Titanium in the Pig Iron Mixer.) Bernhard Osann. (50) Oct. 20.

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The 4 000-Pound Demolition Bomb.* William A. Borden. (46) Dec.

Mining.

Tension of Winding Ropes.* J. Stoney. (Paper read before South Staffordshire and Warwickshire Inst. of Min. Engrs.) (57) Oct. 14.
Inbye Pumping, With Special Reference to the Feuerheerd Pump.* S. H. Cashmore. (Paper read before South Staffordshire and Warwickshire Inst. of Min. Engrs.) (57) Oct. 14.
Pit Shafts.* H. Eustace Milton. (Abstract of paper read before Midland Counties Inst. of Engrs.) (22) Oct. 21; (57) Oct. 21.
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The New Mine Rescue Apparatus.* Henry Briggs. (Paper read before British Assoc.) (11) Oct. 21.
Ventilation in Metal Mines. D. Harrington. (U. S. Bureau of Mines.) (103) Oct. 29.
Heat from Steam Pipe of Pump Ignites Coal in Slope at Springhill, Nova Scotia; How Fire Is Extinguished.* J. C. Nicholson. (Paper read before Min. Soc. of Nova Scotia.) (45) Nov. 3.
Electrical Considerations Which Govern in a Choice of Locomotives for Any Given Class of Service.* H. H. Johnston. (45) Nov. 3.
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Modern Practice in Diamond Core Drilling in the United Kingdom. J. A. MacVicar. (57) Nov. 4.
Shaft Recovery in the North of France.* M. Guerre. (From *Revue de l'Industrie Minérale.*) (57) Nov. 4.
Novel Headgear at a Cape Breton Colliery.* A. Dawes. (Paper read before Min. Soc. of Nova Scotia.) (57) Nov. 11.
Use of Scrapers Underground.* Lucien Eaton. (103) Nov. 19.
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The Discrepancy Between Drilling and Dredging Results. R. G. Smith. (16) Nov. 19.
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Miscellaneous.

The Application of Refrigeration to Big Industries. R. H. Tait. (115) July-Sept.
Recovery of Hydrocyanic Acid and Carbon Disulphide from Coke Oven Gas and Illuminating Gas.* M. Minot. (From *Chimie et Industrie.*) (24) Oct. 29.
How to Follow Up Power Costs.* N. A. Craigue. (9) Serial beginning Nov.
The Metric System of Weights and Measures.* David A. Molitor. (5) Nov.
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Radium Production in America.* H. D. d'Aguiar. (105) Serial beginning Nov. 2.
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Details of Index Number Construction and Comparison of Indices.* E. E. George. (86) Nov. 9.
Industrial Ventilation.* R. L. Gould and E. L. Hewitt. (72) Nov. 17.
L'Evolution et les Progrès de la Mécanique Appliquée. (Evolution and Progress of Applied Mechanics.) M. Drosne. (32) Oct.-Dec., 1920.
Das Materialprüfungswesen in Deutschösterreich. (Method for Testing Material in German Austria.) B. Kirsch. (53) Oct. 14.

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St. Paul City Planning Projects Form Broad Scheme.* (13) Nov. 3.
La Construction des Villes et Cités-Jardins. Rapport de Mission de M. de Heem. (The Construction of Cities and Garden Cities. Report of the Trip of Mr. de Heem.) (30) Aug.
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On the Question of Safety Appliances on Light Railways.* A. Bonnevie. (88) Oct.
On the Question of Economic Production and Use of Steam on Locomotives.* G. J. Churchward. (88) Oct.
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On the Question of Operation of Light Railways, Working Rules and Regulations. F. Level. (88) Oct.
A New Great Northern Dining Car Train.* (12) Oct. 21.
Bowen Gasoline Motor Driven Passenger Car.* (15) Oct. 29.
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Effect of Car Weight and Speed on Coal Consumption.* George S. Chiles. (18) Oct. 29.
The Influence of Rigid Connections on the Distribution of Live Loads on Railway Underbridges.* Conrad Gribble. (21) Nov.
New "Consolidation" (2–8–0 Type) Locomotives for the Western Maryland Railway.* (21) Nov.
The Resignalling of the Liverpool Overhead Railway.* (21) Nov.
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Distributing Expenditures in Track Maintenance.* J. L. Starkie. (87) Nov.
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Avoidable Waste in Locomotive Operation as Affected by Design. James Partington. (55) Nov.
Recent Locomotive Practice on the Caledonian Railway. E. C. Poultney. (12) Serial beginning Nov. 4.
Plan for Electrifying Sections of 11 Railroads.* (15) Nov. 5.
The Requirements for a Modern Car Repair Shop.* H. H. Dickinson and Paul Schioler. (15) Nov. 5.
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The Use of Ordinary Box Cars for Shipping Fruit.* (18) Nov. 12.
Classification and Distribution of Second Hand Rail for Use in Track. (From Report read before Roadmasters and Maintenance of Way Assoc.) (86) Nov. 16.
Magnetic Surveys of Railroad Rails.* (20) Nov. 17.
Locomotive Track Scale Shows Load on Each Wheel.* Carl C. Bailey. (13) Nov. 17.
C. & O. Improves Line and Grades at St. Albans, W. Va.* (15) Nov. 19.
Cedar Hill Yard of the N. Y. N. H. & H. R. R. at New Haven, Conn. (18) Nov. 19.
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Note sur les Nouvelles Règles de Freinage à Main des Trains.* (Note on the New Regulations for Hand-Braking of Trains.) M. Pincemaille. (38) Oct.
Les Ponts Roulants Electriques à Commande Système Ward Leonard, des Ateliers de Réparations de Locomotives de l'Etat, à Sotteville-lès-Rouen.* (Electric Travelling Cranes with Ward Leonard System of Control, in the State Locomotive Repair Shops at Sotteville-lès-Rouen.) (33) Oct. 1.
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Weichen mit gekrümmten Herzstücken.* (Switches with Bent Frogs.) Schwarz. (40) Mar. 6, 1920.
Neuerungen im Eisenbahnoberbau.* (Improvements in Railway Permanent Way.) Jos. Weig. (40) Apr. 3, 1920.
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Der Umbau der Landpfeiler der Stadtbahnbrücke über die Spree am Bahnhof Bellevue in Berlin.* (Rebuilding the Shore Piers of the Municipal Railway Bridge Over the Spree to the Bellevue Station in Berlin.) Kuhnke. (40) Sept. 11, 1920.
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Eisenbetonausführungen im Empfangsgebäude auf Bahnhof Saarburg in Lothringen.* (Ferro-Concrete Construction in the Station Buildings of the Sarrburg Station in Lorraine.) Borchers. (40) Oct. 30, 1920.
Die zweigleisige Johannestahlbrücke im Zuge der Eisenbahnstrecke Hannover-Hamm.* (The Johannes Double Track Steel Bridge in the Hannover-Hamm Railway Division.) Gaede. (40) Nov. 13, 1920.
Die bulgarische Balkanquerbahn von Tirnowo nach Stara Sagora.* (The Bulgarian Trans-Balkan Railway from Tirnowo to Stara Sagora.) Remy. (40) Serial beginning Nov. 27, 1920.
Die Eisenbahnbrücke über das Hollenthor in Newyork.* (The Railway Bridge Over Hell Gate at New York.) Schinkel. (40) Dec. 18, 1920.
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Ueber die Bewegungen der Hauptpfeiler-Köpfe der Trisannabrücke an der Arlbergbahn.* (On the Movements of the Top of the Main Pier of the Trisanna Bridge on the Arlberg Railway.) Leopold Orley. (107) Oct. 29.

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On the Question of Electric Traction (Holland and Great Britain). J. J. W. Van Loeven Martinet. (88) Oct.
On the Question of Electric Traction.* (Switzerland.) E. Huber. (88) Oct.
Trackless Trolleys at Work Abroad.* Walter Jackson. (17) Nov. 12.
Dead Mileage Saving to Pay for New Car Storage Facilities.* (17) Nov. 12.
Double-Truck, One-Man, Two-Man Cars in Milwaukee.* (17) Nov. 26.
Die Madrider Untergrundbahn.* (The Madrid Underground Railway.) O. Jürgens. (40) Sept. 29, 1920.

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Laboratory Control of Road Concrete. H. S. Maltimore. (96) Oct. 27.
Resurfacing and Surface Treating of Macadam Roads. T. J. Wasser. (96) Oct. 27.
A New Method of Making Concrete Roads.* (12) Oct. 28.
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Cause and Correction of Shoving of Asphalt Pavements. Prevost Hubbard. (From paper read before Am. Soc. for Municipal Improvements.) (86) Nov. 2.
The Minneapolis Experimental Wood Block Pavement After 15 Years' Service.* (86) Nov. 2.
Standard Estimate of Cost Form for Road Construction. (Kentucky Assoc. of Highway Contractors.) (86) Nov. 2.
Bureau of Roads' Sub-Grade Drainage Tests Give Interesting Data.* (86) Nov. 2.
A Study of Topeka Mixed Bituminous Pavement Specifications. Roy M. Green. (13) Nov. 3.
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Effect of Age on Strength of Concrete for Highways.* Duff A. Abrams. (96) Nov. 10.
Highway Subgrade Drainage. H. D. Williar. (Paper read before Univ. of Pennsylvania Highway Conference.) (96) Nov. 10.
Reinforcing Steel for Road Use.* Charles W. Geiger. (20) Nov. 10.
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Small Sewage Disposal Plant Operation.* L. F. Bellinger. (36) Nov.
Power Gas from Sewage.* J. D. Watson. (117) Nov.
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Non-Bacterial Population of Sewage Trickling Filters. Charles R. Cox. (13) Nov. 3.
The Operation of Sewage Treatment Plants.* John H. Dunlap. (From paper read before Iowa State College.) (86) Nov. 9.
A New System of Sewage Disposal. (12) Nov. 4.

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Turkey Creek Sewer One of the Largest Yet Built.* Alfred D. Ludlow. (13) Nov. 17.
Open *vs.* Closed Impellers in the Brockton Sewage Pumping Plant. H. S. Crocker. (13) Nov. 24.
Inverted Siphon in Sewer System.* Burt Harmon. (100) Nov.–Dec.

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The Cause and Prevention of Decay in Structural Timber.* R. J. Blair. (5) Nov.
Huge Steel Truss Placed in Chicago U. S. Mail Terminal.* R. F. Imler. (117) Nov.
Precast Ornamental Concrete Units Beautify Theater.* (117) Nov.
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Light Weight Aggregate Economical in Concrete Building.* (13) Nov. 10.
Washington Building Reconstruction—Columns of Lower Story Replaced by Girders.* T. Kennard Thomson. (13) Nov. 10.
The Prince Edward Hotel Foundations.* R. E. W. Hagarty. (96) Nov. 10.
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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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SYNOPSES OF PAPERS

CONTROL OF FLOOD AND TIDAL FLOW IN THE SACRAMENTO AND SAN JOAQUIN RIVERS, CALIFORNIA*

By C. S. Jarvis,† M. Am. Soc. C. E.

This paper briefly reviews the progress made thus far toward flood control on the Sacramento and San Joaquin Rivers, California, and describes the new conditions obtaining along the lower courses of these rivers, which seem to demand the exclusion of the tidal effects from these channels.

The intricate relationship between navigation, irrigation, land reclamation, drainage, water supply, and timber preservation in this district, and the varying salinity of the Upper Bay and lower river channels are discussed, together with proposed methods of controlling the existing menace.

Essentially the paper consists of a presentation of the outstanding features of one of the most important engineering and economic problems in California, with a basic solution proposed and supported by examples and precedents.

The principal purpose of the paper is to demonstrate the necessity for a regulating dam with locks near the mouth of the combined Sacramento and San Joaquin Rivers. Data on areas and storage capacities, average yearly yields, some examples and precedents, etc., are given in an Appendix, which are of suggestive value for the discussion of the problem.

Members who desire a copy of this paper in full are requested to fill out the order blank and forward it to the office of the Secretary. The paper contains 15 pages, including 1 table, and is illustrated by 5 diagrams.

* This paper will not be presented for discussion at any meeting of the Society, but written communications on the subject are invited for distribution and publication with the paper in *Transactions*.

† Capt., 4th Engrs., U. S. A., Camp Lewis, Wash.

PARABOLIC WEIRS*

By F. W. Greve, Esq.†

This paper describes tests of parabolic weirs, which were developed to:

1.—Facilitate the use of a comparatively simple and accurate recorder for measuring the discharge.

2.—Develop a notch that would give a wider range in head for a given discharge than the rectangular opening.

3.—Produce a formula easy of computation.

A series of experiments on seven parabolic weirs was made in the Hydraulic Laboratory of Purdue University, the investigation covering a period of 9½ months. The notches were cut in ⅛-in. brass plates with the exception of one, for which the thickness was $\frac{1}{16}$ in. Three of the weirs were calibrated with the edge of the opening both beveled and unbeveled, while the remainder were tested for the former condition only.

The weirs were placed at the ends of suitable channels from which the discharge passed into a calibrated weighing tank with a capacity of 20 tons. Time was noted on a stop-watch electrically operated by the weighing scales. The elevation head

* This paper will not be presented for discussion at any meeting of the Society but written communications on the subject are invited for distribution and publication with the paper in *Transactions*.

† Asst. Prof. of Hydraulics, Purdue Univ., Lafayette, Ind.

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PAPERS

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on the weir was measured by a piezometer and at least one hook-gauge. A second hook-gauge was used in several of the tests to ascertain what drop, if any, occurred in the water surface within the channel of approach. False bottom and sides were used in the channel to produce a wide range in the velocity of approach.

Calibration tests were also conducted on a recorder that indicated the quantity of water passing through the weir for a given period of time. The device was operated by a float, quantities being indicated on both a chart and a counter. The error in the meter was less than 1 per cent.

The results of the investigation indicate that:

1.—The parabolic weir is one of the best types developed and is adaptable to a wide range of conditions.

2.—The actual rate of discharge can be computed by the formulas:

$$q = hk^2, \text{ and}$$

$$q = C \frac{\pi}{4} \sqrt{2\,g} \sqrt{2\,p}\, h^2$$

where q is the actual rate of discharge, in cubic feet per second; h is the elevation head, in feet; k is a constant for any given weir with beveled notch; C is the coefficient of discharge; $\pi = 3.1416$; $g = 32.16$ ft. per sec. per sec.; and p is a constant in the equation of a parabola, $x^2 = 2py$.

3.—The parabolic weir is adaptable to a simple and accurate meter for measuring the discharge.

4.—The values of the constant, k, can be accurately predicted for any given value of p.

Members who desire a copy of this paper in full are requested to fill out the order blank and forward it to the office of the Secretary. The paper contains 24 pages, including 4 tables, and is illustrated by 13 diagrams and 2 half-tones.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

PROGRESS REPORT OF THE SPECIAL COMMITTEE TO CODIFY PRESENT PRACTICE ON THE BEARING VALUE OF SOILS FOR FOUNDATIONS*

TO THE AMERICAN SOCIETY OF CIVIL ENGINEERS:

Your Special Committee appointed "To Codify Present Practice on the Bearing Value of Soils for Foundations, and Report upon the Physical Characteristics of Soils in Their Relation to Engineering Structures" respectfully submits the following statement of its activities.

During 1920, three meetings have been held, the minutes of which have been submitted to the Society. Although the work of the Committee has not involved obligations or large expenditures, it has been conducted for the past year or two without financial support by the Society. A budget for the coming year, however. has been submitted to the Board of Direction which, if approved, will give the Committee much needed funds to continue its studies, if the Society so desires.

Your Committee here records with regret, the resignation of Edwin Duryea, M. Am. Soc. C. E., because of ill health. From the first meeting to the present time he has sustained his interest by participating in the Committee's complex problems and their development. His ability and assistance are here formally recognized by his colleagues who extend to him their hope for his early restoration to health.

The Committee has followed its past practice of presenting its report in the form of appendices, of which this report contains six, as follows:

Appendix I presents definitions of soils in order to stimulate discussion of local soils and their characteristics. Throughout the West there are a number of soils with which the members of the Committee are not familiar, and they would like to hear from members of the Western districts in order to include all soils of importance.

In Appendix II the mineral composition of soils has been considered. This study, taken from *Bulletin No. 85*, U. S. Department of Agriculture, is a synopsis of the subject from the agronomist's point of view, and is helpful because of the indirect effect of composition on the physical characteristics of soils. It may be noted that soils in the arid and glacial areas possess a great variety of minerals, and soils in the humid areas have little variation in composition.

Appendix III describes a simple procedure for ensuring uniformity of practice in regard to color of soils, in deference to the natural practice of describing soils by their color.

* Presented to the Annual Meeting, January 19th, 1921. Previous Progress Reports of this Special Committee have been published in *Proceedings*, Am. Soc. C. E. (Papers and Discussions), February, 1915, p. 491; March, 1916, p. 343; August, 1917, p. 1171; and August, 1920, p. 905.

Appendix IV is a revision and rearrangement of the important factors in soil classification in which the controlling influence and importance of capillary water are emphasized. This water is constant. Water of gravitation will vary with the seasons or with topography. The water content is an important factor in any scheme of soil classification. The texture, or size-grades, of soils is equally important, as extremes in size-grades control the type of soil. For instance, it must also be admitted that the influence of colloidal clays is quite pronounced in tension and shearing tests. This may possibly lead to important results. Further experimentation is desirable. Porosity relates to the total pore space, whether occupied by water or not. It differs from absorption, which refers to the quantity of water taken up. It, also, is a factor of importance.

To avoid any further misunderstanding as to terms, Appendix V is submitted for discussion. It contains definitions of settlement, allowable loads, displacement, bearing capacity, etc. These terms are somewhat vague, and the Committee thought it desirable to submit definitions of them for consideration.

Appendix VI describes laboratory apparatus which was developed in 1914 and 1915. The Committee commenced its laboratory work with a standard apparatus used by agronomists, but this was subsequently found to be unsuited for the purposes of the Committee, and was replaced by instruments developed for the needs of the case as occasion required. This has entailed much study and labor, the usual accompaniment of original research work.

There is one outstanding feature to which your Committee would direct attention. In the development of screening and separating methods the substitution of a standard centrifugal force for gravity facilitates and improves the processes. For example, the use of the high-speed centrifuge has made possible the precipitation of microscopic and ultra-microscopic particles. Through this means the value of colloid clay has been identified and its collection simplified.

The Committee also is studying the engineering formulas now in use, and hopes to present a workable general formula at a later date.

It is extremely gratifying to your Committee to find, during the past year, an evident recognition of the importance of this problem, which is of a pioneer nature. The American Institute of Mining and Metallurgical Engineers has recently organized a similar committee. The National Research Council, the Bureau of Public Roads of the U. S. Department of Agriculture, the Federal Highway Council, and many universities and colleges have taken up divisions of the subject, and it is anticipated that satisfactory results will be available earlier than could otherwise be expected.

With your approval, the Committee will continue its work during the coming year.

Respectfully submitted,

ROBERT A. CUMMINGS,
Chairman.

COMMITTEE:

ROBERT A. CUMMINGS, *Chairman,*
WALTER J. DOUGLAS, *Secretary,*
E. G. HAINES,
ALLEN HAZEN,
J. C. MEEM.

APPENDIX I.

Definitions of Soils.

Your Committee has given further consideration to definitions for soils that will combine common and practical ideas with controlling physical factors. The difficulty is emphasized, however, through the frequent confusion of the physical state of the soil material with the soil as a mass. For example: "Quicksand" is a physical state of water and granular material, rather than a type of soil.

A general discussion of soils met with in construction work by members of the Society might develop further suggestions for definitions. For this reason some common definitions of soils are presented, as follows:

Alluvium.—The finer deposit of earth, sand, gravel, and other transported material, usually occupying the lower parts of valleys and great rivers, which has been washed away and thrown down by rivers, floods, or other causes, on land not permanently submerged.

Bog.—A quagmire covered with grass or other plants. It is defined by marsh and morass, but differs from a marsh as a part from the whole. Wet grounds are either bogs, which are the softest and too soft to bear a man; marshes or fens, which are less soft but very wet; or swamps, which are soft, spongy land on the surface, but sustain man or beast, and are often mowed. A little elevated spot or clump of earth in marshes and swamps, piled with roots and grass—this is a common use of the word in New England.

Clay.—A general name for cohesive soils. The name of certain substances which are mixtures of silex and alumina, sometimes with lime, magnesia, alkali, and metallic oxides. A species of soil which is firmly coherent, weighty, compact, and hard when dry, but stiff, viscid, and ductile when moist, and smooth to the touch, absorbs water greedily but not readily, diffusible in water and, when mixed, not readily subsiding in it.

Clay is also defined as the material resulting from the decomposition and consequent hydration of the feldspathic rocks, especially granite and gneiss, and of the crystalline rocks in general. As thus formed, it almost always contains more or less sand, or silicious material, mechanically intermixed. After this has been separated, the clay itself is found to consist of a hydrated silicate of alumina, but it is not yet positively determined that there is one definite combination of this kind constituting the essential basis of all the substances to which the name clay is applied. All clays contain hygroscopic water which may be expelled by heating to 212° Fahr., but they also contain water in chemical combination, and when this is driven off by ignition the clay loses its plasticity and shrinks in volume, neither of which can be restored by the addition of water. The lime and other impurities present in ordinary clay render it to a certain extent fusible. The purer varieties are refractory and are known as fire-clay. The plasticity of clay is of great importance, as without this quality it could not be easily worked into the various shapes for which it is used. On what condition it depends has not as yet been clearly determined. Clay is any mixture of silica and alumina in a finely pulverized condition; a mixture of granular materials and a colloid.

Clay is also defined as mixtures of minerals of which the representative members are silicate of aluminum, iron, the alkalis, or the alkaline soils. The hydrated

aluminum silicate, kaolin, is the most characteristic of these. Some feldspar is usually present. The grains of these minerals may show crystal faces (especially in the case of kaolins), but more commonly they are of irregular shapes; upon most of these grains is an enveloping colloid coating. This is mainly of silicate constitution, but may consist partly of organic colloids, of iron, manganese, and aluminum hydroxides, and of hydrated silicic acid. Quartz grains, which are generally present, do not have the colloidal coating, or have it in much less degree. Almost any mineral may be present in clays, and modify the properties somewhat. The combination of granular materials and colloids is in such proportion that when reduced to sufficiently fine size (by crushing, sifting, washing, or other means) and properly moistened with a proper quantity of water, plasticity is developed. If the colloid matter is in excess the clay is considered to be very plastic, fat, or sticky, but if the granular material is in excess it is called sandy, weak, or non-plastic.

The colloid matters in clay are non-crystalline, hydrated, gelatinous, aluminum silicates, organic colloid, gelatinous silicic acid, and hydrated ferric oxide. Rarely, aluminum hydrate may also be present.

Detritus.—A mass of disintegrated rock material, loose or uncompacted, worn and broken off from larger solid bodies, either water-worn or angular, and reduced by attrition to relatively small portions, as diluvial detritus. The term is especially applicable to a material which would be a breccia, or conglomerate, if consolidated into rock. When the portions are large, the word débris is used. More comprehensively, any broken or comminuted material worn away from a mass by attrition; any aggregate of loosened fragments or particles.

Diluvium.—A superficial deposit of sand, loam, gravel, pebbles, or other coarse detrital material wherever found, caused by the deluge or ancient currents of water.

Drift or Glacial Drift.—A heap of loose detrital material, fragments of rocks, boulders, sand, gravel, or clay, or other soil driven together, or a mixture of two or more of these deposits, resting on the surface of the bedrock.

Dust.—Fine, dry particles of soil, or other matter, so reduced to powder or attenuated that it may be raised or wafted by the wind; powder; fine soil.

Earth.—The particles which compose the mass of the terraqueous globe, but more particularly the particles which form the fine mold on the surface of the globe; or any indefinite mass or any portion of that matter.

Glacial Drift.—See Drift.

Gravel.—Small stones or fragments of stone or very small pebbles larger than the particles of sand, but often mixed with them.

Gravel may be caused to cohere by infiltrated calcareous or silicious matter, or by the effect of such infiltration combined with that of pressure, and is sometimes called natural concrete, and indurated gravel, conglomerates, and breccia.

Grit.—Angular, rough, hard particles of sand or gravel in a loose form. The term is also sometimes applied to this material in a combined, solidified form; for example, certain classes of sandstone from which grindstones are made.

Ground, or Filled Ground (Made Land).—The surface of land, or upper part of the earth, without reference to the materials which compose it. Ground is

applied to soil, indifferently, but it is never applied to the whole mass of the earth, nor any portion of it when removed. We never say a shovelful of ground.

Hardpan or Pan.—The hard stratum of consolidated soil underlying the surface soil; loess of 50 to 75% silt and up to 15% clay.

Loam.—A natural mixture of sand and clay with oxide of iron; a species of soil of different colors, whitish, brown, or yellow, readily diffusible in water. A clay soil containing more or less of carbonate of lime, and consequently effervescing with an acid.

Marsh.—A tract of low land, usually or occasionally covered with water, or very wet and miry, and overgrown with coarse grass or detached clumps of sedge; a fen. It differs from a swamp which is merely moist or spongy land, often producing valuable crops of grass. Low land occasionally overflowed by the tides is called salt marsh or tidal marsh.

Mold or Mould.—A fine soft soil, such as constitutes garden or vegetable mold.

Muck.—The term muck as commonly indicated by engineers and contractors generally means any excavated material removed or to be removed from an excavation. Hence, the allied term, "muckers" applied to laborers who handle broken rock, as well as earth or other excavated material.

A wet slimy mass of decaying or putrified vegetable matter; swamp muck; imperfect peat; the less compact variety of peat, especially the paring or tuff overlaying peat.

Mud.—Moist and soft soil of any kind, whether produced by rains on the earth's surface or by ejections from springs and volcanoes or by sediment from turbid waters; such material as is found in marshes and swamps, in the beds of rivers and ponds, or in highways after rain.

Ooze.—Soft soil mud or slime so wet as to flow gently, or easily yield to pressure.

Peat.—A brown soil of vegetable origin consisting of partly decomposed roots and fibers, more or less saturated with water. It is found in every stage of decomposition, from the natural wood to the completely black vegetable mold. It is produced under various conditions of climate and topography, and is of considerable importance in certain regions as fuel. Peat is very spongy, and contains a large quantity of water near the surface; the deeper down it is taken, the more compact it is. It is formed of vegetable matter undergoing decay and in some respects it is the modern representative of the coal of the earlier geological epoch.

Pebbles.—Roundish stone of any kind, from the size of a nut to that of a man's head.

Quagmire.—A soft, wet, swampy land, which has a surface firm enough to bear a person, but which shakes or yields under the feet.

Quicksand.—Sand easily moved or readily yielding to pressure; loose sand, abounding with water, such as a movable sand-bank in a sea, lake, or river; a large mass of loose or moving sand mixed with water formed on many sea coasts, at the mouths and in the channels of rivers, etc.; sand supersaturated with water temporarily, and when under pressure acting as a fluid.

Rock.—A large mass of stony matter, either bedded in the earth or resting on its surface.

Rock-Flour.—Microscopic sand, or rock pulverized to a degree of fineness resembling powder or dust.

Sand.—Any mass or collection of fine particles of stone, particularly fine particles of silicious stone, but not strictly reduced to powder or dust; dune sand.

Shale.—A fine-grained, indurated, clayey rock having a slatey structure.

Silt.—Fluvial sediment of mud or fine soil deposited from running or standing water.

Soil.—The unconsolidated veneer covering the rock crust of the earth. The upper stratum of the earth; the mold, or that compound substance which furnishes nutriment to plants, or which is particularly adapted to support and nourish them.

Till.—*See* Drift.

APPENDIX II.

Mineralogical Composition.

Your Committee has given further consideration to the influence of the mineral composition of soils. The failure of certain soils by decomposition or crushing of particles, has led to a fruitful field for study in the relation of the mineral composition to the physical factors. In this connection "sedentary" soils frequently retain the original minerals unchanged, whereas in "transported" soils the minerals present are those that most tenaciously resist wearing and weathering.

Mineralogically, the soil varies with its chemical composition; it may be calcareous, alkaline, ferruginous, inicallous, silicious, etc. The presence of oxidizable sulphides may cause heating and disintegration of the soil when exposed to the air. According to Dr. Donald F. MacDonald this happened locally in the Gaillard Cut, Canal Zone, Panama. It sometimes happens in coal and culm piles.

The leaching out of soluble sulphates or other salts may cause clays to disintegrate rapidly and slide. Calcareous material, if present, may also be dissolved out of clays, causing them to lose cohesion.

It is hoped that those who are mineralogically expert will present constructive criticism, or their experience with soil failures that can be attributed to the mineral nature of the soil.

The following is a synopsis of a study, by Dr. G. N. Coffey, of twenty-five surface soils from the Coastal Plain, Piedmont Plateau, and Limestone Sections of the United States, as found in *Bulletin No. 85,* U. S. Department of Agriculture.

Characteristics of Surface Soils.

A.—Arid soils have a large percentage of minerals other than quartz.

B.—Humid soils, with the exception of orthoclasse and microline, have less abundance of feldspars.

C.—The influence of topography on the surface soil is often very marked. In mountainous regions erosion allows only a thin mantle of soil to accumulate. The minerals show a very slight alteration indicating that weathering has not been acting for a very long period. In the plateau regions the surface is not broken, giving rise to less erosion, but more advanced decomposition.

D.—Limestone soils have little variations in minerals, and such as are present occur in very small particles.

E.—In the unconsolidated water-laid deposits of the Coastal Plain a high percentage of quartz is abundant.

F.—Soils formed from glacial material are characterized by a relatively large percentage of minerals other than quartz, especially in the sands. The grains are only slightly rounded.

G.—In loessial soils about 75% of the soil mass consists of silt. The grains are mostly angular, with some fairly well rounded.

H.—The total number of minerals found in the twenty-five surface soils is 34, the average number present in a sample being a fraction more than 13 (Table 1).

J.—There appears to be a considerable variation of mineralogical composition. Soils usually have a greater variation than rocks, since they are the dispersed products of rocks through degeneration and decomposition.

K.—The characteristics of the different minerals are as follows:

Quartz is the most abundant mineral occurring in every sample. Quantitatively, quartz constitutes from 50 to 95% of the surface soil, the average being 83%, but

TABLE 1.—MINERAL ANALYSIS OF TWENTY-FIVE SURFACE SOILS, FROM BULLETIN 85, U. S. DEPARTMENT OF AGRICULTURE.

	Specific gravity.	Percentage of Occurrence.			Usual color.
		Abundance.	Less abundance.	Absence.	
ANHYDROUS SILICATES:					
Feldspar Group:					
Orthoclasse	2.7	56	24	20	Reddish
Microline	2.55—2.57	40	40	20	"
Plagioclasse	2.61—2.76	20	32	48	Gray or white
Andesine	2.68	16	8	76	" " "
Oligoclasse	2.7	12	16	72	" " "
Albite	2.56—2.62	8	12	80	Reddish
Labrodorite	2.7	8	4	88	
Amphibole Group:					
Hornblende	2.9—3.5	48	44	8	Dark green and black
Actinoite	2.9—3.5	0	4	96	Bright green
Mica Group:					
Biotite	2.7—3.2	32	52	16	Brown, black or green
Muscovite	2.7—3.2	24	56	20	White to gray
Sericite	2.85	4	0	96	" " "
Phlogopite	2.7	0	4	96	Copper color
Epidote	3.3—3.5	32	64	4	Greenish
Pyroxene	3.2—3.6	8	16	76	White, green or black
Augite	3.2—3.6	8	0	92	Green to black
Garnet	4.0—4.1	8	32	60	Var. usually red
Tourmaline	2.94—3.2	4	80	16	" " black
HYDROUS SILICATES:					
Chlorite	2.65—2.96	8	72	20	Usually dark green
Serpentine	2.2—2.7	0	4	96	Greenish
OXIDES:					
Quartz	2.65	100	0	0	Var. white, red, black
Magnetite	5.16—5.18	4	20	76	Iron black
Ilmenite	4.5—5.0	4	4	92	" "
Hematite	4.8—5.3	4	0	96	Iron black to deep red
CARBONATES:					
Calcite	2.5—2.72	4	4	92	White or colorless
PHOSPHATES:					
Apatite	3.15—3.16	0	48	52	Green or brown
Zircon	4.6	4	84	12	Pale yellow to red, brown
Rutile	4.18	0	68	32	Reddish brown
Fluorite	3.01	0	16	84	
Titanite	3.5	0	8	92	
Staurolite	3.65	0	4	96	

it constitutes less of silt than of sand, indicating that a smaller percentage would be found in clay. The quantity of quartz is greatest in surface soils derived from rocks that have undergone the most attrition and decomposition.

Quartz occurs in largest quantities in the southeastern part of the United States.

Epidote is the next most common mineral, followed by hornblende and the two feldspars—orthoclasse and microline. The latter were not found in the reddish soils.

Of all the varieties of minerals, feldspar occurs next in abundance to quartz. The plagioclasse feldspars—labrodorite, andesine, oligoclasse, and albite—do not appear where the soil is subjected to the greatest leaching and attrition.

The micas—biotite and muscovite—occur rather frequently. Chlorite, zircon, tourmaline, and rutile are all common minerals, but are seldom abundant, the three latter being hard-weather resisting minerals.

The apparent reason for the small occurrence of the iron minerals—magnetite and hematite—in surface soils is due to the fact that organic matter attacks iron very readily. Iron is commonly found in rocks.

L.—The mineralogical determinations were applied solely to the twenty-five surface soils from a composite of ten samples. Since the surface soils in the southeastern portion of the United States are more sandy than the subsoils, a large percentage of quartz might be expected in the latter. It is also probable that the larger percentage of organic matter in the surface soils would cause more leaching to take place, and that the subsoil would not only show a smaller percentage of quartz but also a greater variety of minerals.

A greater percentage of minerals is found in the north, due to the glacial action of grinding down of rocks containing various materials.

APPENDIX III.

The Color of Soils.

Your Committee adheres to the statement made in a previous report that the color of the soil has a very limited physical significance. In deference, however, to the universally established habit of referring to color in describing soils, a simple procedure is here presented to ensure uniformity of practice.

Although the color of soils is influenced by the mineral composition, drainage, and content of organic matter, the time color is that which it possesses when exposed under field conditions. Soils that vary in color may be described as mottled.

In positioning a soil in the color scheme, it is desirable to avoid precise variations. The method developed, however, introduces accurate comparisons by adapting the idea of whirling disks of various colors. Table 2 presents the results of experiments to determine the composition of fifteen standard colors. These are sufficient for the range of ordinary soils.

The seven columns of percentages in Table 2 have reference to seven paper disks of standard colors. Each disk, 5 in. in diameter, is slit from the center to the circumference, in order to enable other color disks to slide between and expose the percentage of color desired, as seen in Fig. 7. The percentages are obtained by

markings on a 5½-in. disk, at the back. This enables a quick and ready adjustment of color disks adaptable to any color of soil.

The color disks have holes 2 in. in diameter in their centers, to fit over the clamps on the top flange of an open brass cup. In this cup is placed the sample of soil, and it is fastened concentrically with the shaft of a small electric fan motor. The color disks and sample are rapidly rotated by the operation of the motor in a horizontal position (see Fig. 7). The varying percentages of the exposed color disks and the color of the sample of soil are thus easily comparable. When the color of the soil and of the rotating disk match, the percentages may be read and identified in Table 2.

TABLE 2.—COMPOSITION OF FIFTEEN STANDARD COLORS.

Color of Soil.	Percentages of Standard Colors.						
	Black.	White.	Red.	Orange.	Yellow.	Green.	Blue.
Black:	100	..	..	..	..	..	..
Blue:	..	5	..	..	..	13	82
Light Blue	..	22	..	..	..	22	56
Gray:	81	16	..	..	..	..	3
Yellowish	32	17	5	14	25	..	7
Drab	44	41	..	9	6	..	..
Yellow:	..	10	..	..	90	..	..
Reddish	..	13	44	..	43	..	..
Red:	..	..	90	10	..	..	..
Purplish	..	2	77	..	..	..	21
Light	..	5	75	20	..	..	..
Dark	80	13	..	..	..	..	7
Brown:	75	5	15	..	5	..	..
Light	46	4	..	26	24	..	..
Dark	88	..	..	6	6	..	..

APPENDIX IV.

Revised Soil Classification.

Your Committee submits a revision of the proposed classification of soils to which has been added a further sub-division termed "colloids", for the determination of which a centrifuge of very high rotating speed is necessary (about 40 000 r.p.m.). The Committee is not at present in a position to enter into details as to this class of material, but tension experiments conducted upon mixtures of the "colloids" and Ottawa sands have shown such surprisingly high strength as to indicate its great importance. In fact, it now appears that this element alone, or together with water, may account in a large measure for the cohesiveness of soils. The subject is to be further followed up by the Committee, and reported on later.

"Structure" has been changed to refer to the arrangement of the soil mass on bedding, instead of the particles. It is an important factor.

"Porosity" is defined as the equivalent of density or percentage of total pore space. It is an important physical factor, and often determines whether the soil particles are arranged loosely or compactly.

The water in soils may be either of a permanent or of a temporary nature. The permanent water above the water table is due to surface tension and is the capillary water content. Its determination is very important, for it resists the action of gravity and cannot be drained. The constant "dampness" of soils is traceable to this characteristic. The size and arrangement of the pores and the texture of the soil have controlling influence over the quantity of capillary water present. Excess water, or the water of gravitation, is that which responds to the action of gravity and is flowing to a lower level. Such water will vary with the seasons. The phenomena of slides are frequently due to excess water content. Your Committee will continue its study.

Revised Scheme of Classification.

Source of Material.—The following sub-divisions of sedentary and transported soils are recognized as representing the first factor in the divisions of soil classification:

Sedentary soils:
- Residual (formed in place);
- Cumulose (accumulated organic matter).

Transported soils:
- Colluvial, or gravity laid;
- Alluvial, or water laid, by streams, lakes, or oceans;
- Aeolian, or wind laid;
- Glacial, or ice laid.

TABLE 3.—Revised Size-Grades.

Separation methods.	Size-grades.	Diameter of Circular Openings, in Millimeters. Range of size-grades. Finer than	Coarser than
By count and perforated plate screen........	Stones, coarse..........	256.0	128.0
	medium..........	128.0	64.0
	fine..........	64.0	32.0
	Pebbles, coarse..........	32.0	16.0
	medium..........	16.0	8.0
	fine..........	8.0	4.0
By wire screens..........	Grits, coarse..........	4.0	2.0
	medium..........	2.0	1.0
	fine..........	1.0	0.5
	Dust, coarse..........	0.5	0.25
	medium..........	0.25	0.125
	fine..........	0.125	0.0625
By electrification..........	Flour, Coarse..........	0.0625	0.0312
	medium..........	0.0312	0.0156
	fine..........	0.0156	0.0078
	Powder, coarse..........	0.0078	0.0039
	medium..........	6.0039	0.0019
	fine..........	0.0019	......
By super-centrifuge..........	Colloids..........		

Mineral Composition.—This relates to the evident abundance of minerals in the composition of soils.

Structure.—This refers to the natural occurrence of the soil in beds, masses, pockets, or stratified in layers, and the dip of same, and occasionally by cracks, fissures, etc.

Porosity.—This refers to the arrangement of the particles of the soil. It is the density or percentage of total pore space, and is also an important factor.

Water Content.—This refers to the volume of pore space occupied by water. It is an important physical factor.

Texture.—This refers to the relative range ratio and mid-grain size, as determined from plotting the mass diagram of a granulometric analysis.* It is constant and is recognized as an important factor.

Variations in the shape of particles may modify the textural class, such as, flat shaley, slaty, sharp, angular, rounded, corroded-surface grains or fragments of rocks.

The revised size-grades of particles have been tentatively grouped, as in Table 3.

APPENDIX V.

Definitions of Settlement, Allowable Load, Etc.

Settlement.—Settlement is defined as the change of horizontal plane of any part, or all, of a structure, occurring after the beginning of construction. Some settlement occurs before the completion of the structure, and some continues for a time after completion. After reaching a state of rest, other work near-by may influence the soil bearing the structure and cause vertical motion again to take place. This may occupy but a short period of time, after which the structure again comes to rest. Such vertical motion, or settlement, is due to a number of causes. The excavation of the soil and the preparation of the bed of the foundation disturbs a portion of the grains at the surface of the soil, and they become somewhat loosely associated. The application of weight to soil in that condition naturally compresses it. The excavation of the soil also removes a considerable weight from the plane at which the foundation is to be started. The removal of that weight permits certain soils to swell or increase in volume due to loss of the restraining influence of the soil itself, thereby becoming less compact than before the excavation. This latter part is negligible from an engineer's point of view.

The building of the foundation has a tendency to compress the soil to, or even exceeding, the degree to which it was originally compressed. The addition of more weight to the foundation than that of the soil removed for its preparation at once increases the compressive stresses, and reduction of volume and consequent settlement take place up to the yield point of the soil for such compression. If the load is increased beyond that limit, displacement of the soil takes place due to the crushing of the grains, or actual movement of the grains from their original position. All these effects are accompanied by a settlement of the structure. In brief, soil acts in much the same manner as other elastic bodies, within certain limitations.

These different movements or compressions in any soil where it is possible to

* See Progress Report presented January 21st, 1920; *Proceedings*, Am. Soc. C. E., August, 1920, p. 912.

build without the use of piles, or in other words, where there is not an excess of water causing a more or less viscous condition of the soil, are illustrated by a fairly regular form of compression curve. Such a curve shows a relatively large compression at the beginning of the application of the load, diminishing rapidly as the load is applied, then a considerable increase of weight with a fairly regular compression, until finally rapid break-down occurs as displacement or crushing of the grains of the soil takes place.

Even in wet ground, and using piles, much the same form of compression curve is obtained. There is relatively little resistance at the beginning of the curve, increasing as the soil is compacted and the excess water is driven out. As the grains take an elastic bearing, there is a quite wide range of safe compression. This is lost on overdriving, because actual displacement of a considerable quantity of the surrounding soil results, which is thus somewhat restored to its original condition and further compression is necessary to develop its elastic resistance. The resistance is also often restored by a period of rest, during which there is a readjustment of the grains. Such a readjustment also takes place in the grains in the case where piles are driven by jacks, and it is found that if the jacks are released without restraining the piles, further penetration is necessary to secure the same degree of resistance. All these things point to a high degree of elasticity of the soil considered as a mass.

Settlement, as herein defined, should not be confused with the shrinkage of a mass of soil in a loose state from its own weight or by the action of the weather.

Allowable Load.—The allowable load is, in most cases, determined by the question of settlement and the effect of such settlement on the proposed structure. It is obvious that for certain types of structures the settlement should be kept to an absolute minimum, because such settlement, or at least unequal settlements in different parts of the structure, will cause physical damage. For example: if a highly ornamental cut-stone building settled unequally in different parts, it might be much disfigured and injured by reason of spalling or cracking. On the other hand, an elevated railway carried by independent piers may settle considerably at one or more of the bearings with no evidence thereof whatever, except as indicated by the grade of the track. Structures on pile foundations, particularly around harbor and river work, almost invariably settle, but usually they are of a type and designed so that a reasonable amount of settlement is not injurious.

It is believed that the limit of allowable load should be based on some definite portion of that part of a compression diagram showing a practically uniform rate of compression with increase of load. The amount of such percentage should vary with the type and importance of the structure. Manifestly, structures such as reservoirs containing fluids, where settlement would cause cracking and leaks, should be limited to a much smaller percentage of the ultimate load than the column piers of an elevated railroad, isolated monuments, etc.

The possibility of water reaching the soil beneath a foundation should receive careful consideration. Certain soils, such as sand and gravel, are not materially affected by saturation, but are readily eroded by flowing water. On the other hand, clay is not eroded by flowing water, but the surface is readily softened and, when so softened, is plastic and flows under pressure.

Subject to the above considerations, it is the opinion of the Committee that, for the greater number of types of structures, the safe bearing value of soil should be limited to one-half the value, shown by a compression diagram, between the point where the soil is merely compacted and that where displacement begins. That value can be modified as needed, either way, depending on the character and importance of the structure proposed. Such a value agrees fairly well with common practice and usual soil conditions as they have been observed.

It should be noted that allowable load, as herein defined, may be regarded as a function of settlement, whereas bearing capacity may be regarded as the ultimate load the soil will bear without displacement, and displacement is the ultimate measure for settlement. With this in view, the Committee proposes to submit to the membership a blank form on which to record observed settlements and loads, with the desire that such records be submitted to it for compilation.

Code for Soil-Testing Apparatus.—The notes on the drawings submitted and published with the 1920 report* on the use of the soil-testing apparatus appear to be fairly clear as far as dry soils are concerned. The discussions of the Committee seem to indicate, however, that certain phases of the case should be further considered, such as that of wet ground. To use the apparatus proposed in wet ground at all, it would be necessary to unwater the soil, at least until the time the apparatus was in place, because it is presupposed that the soil beneath the bearing-plate is dressed to a fairly smooth contact with the bearing-plate. This could not be done under water, particularly if the soil contained stones or gravel. A test on soil, however, should represent the value of that soil under its actual working conditions, and if the soil is unwatered for the purpose of placing the apparatus, the water should be allowed to return before making the test.

If the test is made on wet soil without special precautions, using the apparatus described, it is believed that there would be a squeezing out of the soil between the edges of the bearing-plate and the tile which surrounds it. It is suggested that this might be overcome in a manner similar to that used in a press for fruit pulp by spreading a strip of burlap, or other similar material—which would permit the water to pass through and retain the grains of the soil—over the surface before placing the bearing-plate, tile, and back-filling. It is not believed that the burlap would have any appreciable effect on the recorded bearing value of the soil.

Under the conditions described, however, there would be a possibility of the surrounding soil rising from the pressure exerted by the compression plate. The recording gauge attached to the side of the pit, or the compression post itself, therefore, should be referenced in some manner to the elevation of a point beyond the zone of influence. This might be conveniently done with an engineer's level.

Your Committee has developed a small type of soil compression apparatus (Fig. 1) based on the principle of penetration. The instrument has had considerable use and is believed to give consistent results. It is comparable in its action to the larger type of field-testing apparatus submitted in the 1920 report of the Committee, and is designed to give the amount of penetration for a given pressure on 1 sq. in. (in circular form) in any given length of time, the larger type of apparatus having been designed to test an area of 1 sq. ft.

* *Proceedings*, Am. Soc. C. E., August, 1920, p. 905.

In this connection attention should be called to the fact that all such tests are relative only, the penetration not varying inversely with the area on account of the pyramiding effect of soil stresses in compression. This is equally true of foundations in general. In the case of both the large and small compression machines, bearing-plates of varying areas may be used to suit soil conditions.

APPENDIX VI.

Laboratory Apparatus.

The Committee has not heretofore entered into the question of laboratory equipment. A considerable amount of work, however, has been done for the purpose of grading soils and soil materials. The equipment may be divided into the following classes:

1.—Centrifugal separator (not now in use).
2.—Centrifugal elutriator (not now in use).
3.—Centrifugal compactor.
4.—Tension and shearing apparatus.
5.—Sample washing machine.
6.—Centrifugal screening apparatus.
7.—Schultze elutriator.
8.—High-speed centrifuge.
9.—Rotating color disks.

Illustrations of these machines are presented herewith, although some have been abandoned as in the case, for example, of the centrifugal separator (Fig. 2) which illustrates the principle of separating the fine material in flowing water by the application of centrifugal force, and the centrifugal elutriator (Fig. 3), which is a later development of the same principle in combination with the ordinary elutriator, but arranged horizontally so that the water flows toward the axis of rotation. Fig. 3 also shows the centrifugal compactor, which was developed in order to obtain uniform density in the packing of samples of granular material to be used in tests.

Fig. 4 illustrates the apparatus finally developed for making tension and shearing tests of soil samples under varying degrees of compaction. Considerable time and thought was expended on the method to be adopted for sampling soils, various types of equipment and containers being devised. However, the Committee finally came to the conclusion that the simple method of using a sharpened shell to dig into the soil to remove it, was the most satisfactory solution for small samples. For more accurate and reliable tests, however, it is desirable to take as large a volume as possible. For instance, a unit of 10 to 20 cu. ft. should be removed, and the weight, porosity, water content, etc., determined.

Fig. 5 and the diagram, Fig. 9, show the machine which was developed for washing samples. Figs. 6 and 7 illustrate the centrifugal screening apparatus developed for separating the size-grades without the necessity of drying the sample, thereby avoiding the errors resulting from such drying due to the caking of the clay on the granular material.

Fig. 8 shows the ordinary Schultze elutriator, as arranged in the laboratory for trial tests.

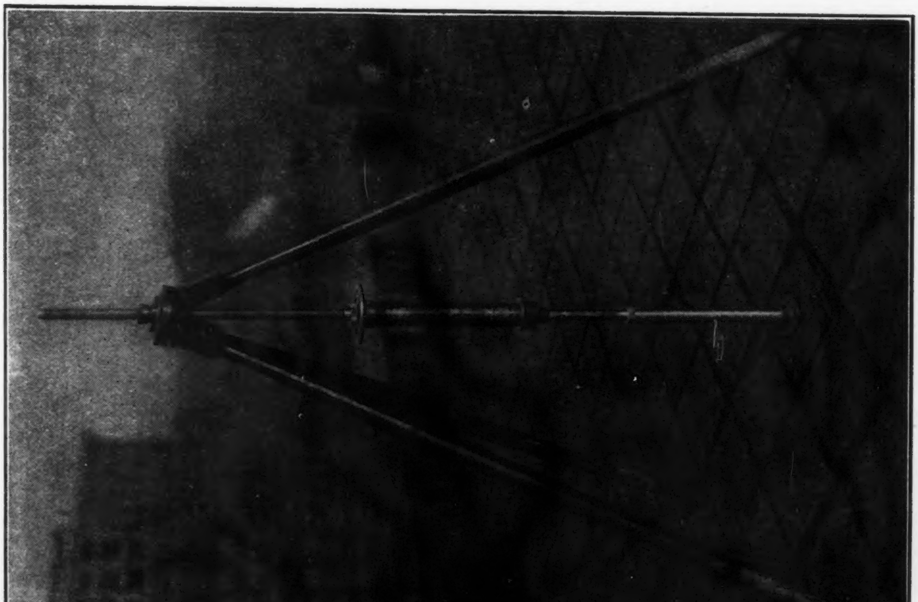


FIG. 1.—SMALL TYPE OF SOIL TEST APPARATUS.

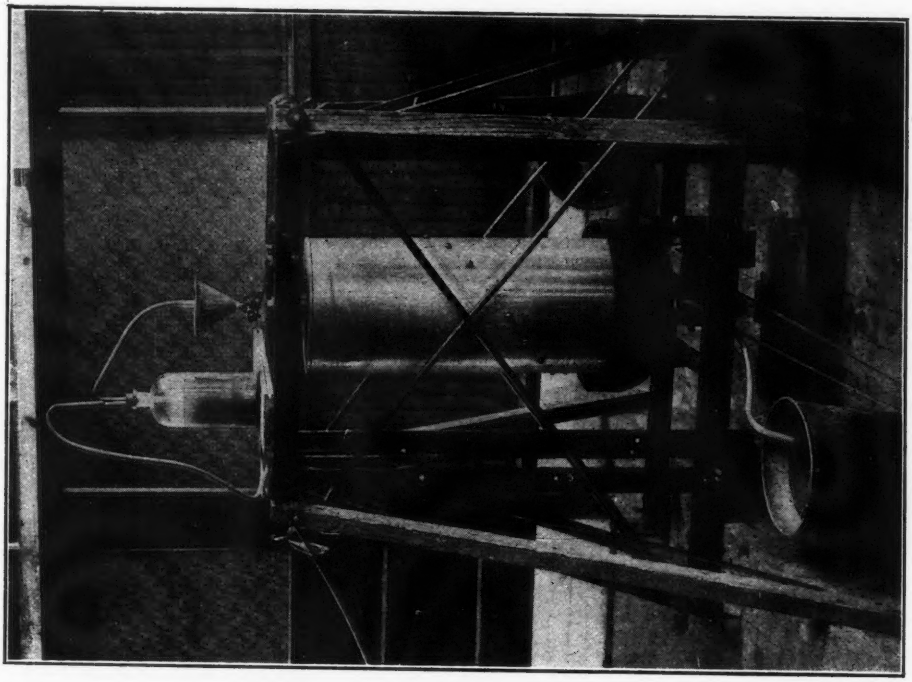


FIG. 2.—CENTRIFUGAL SEPARATOR.

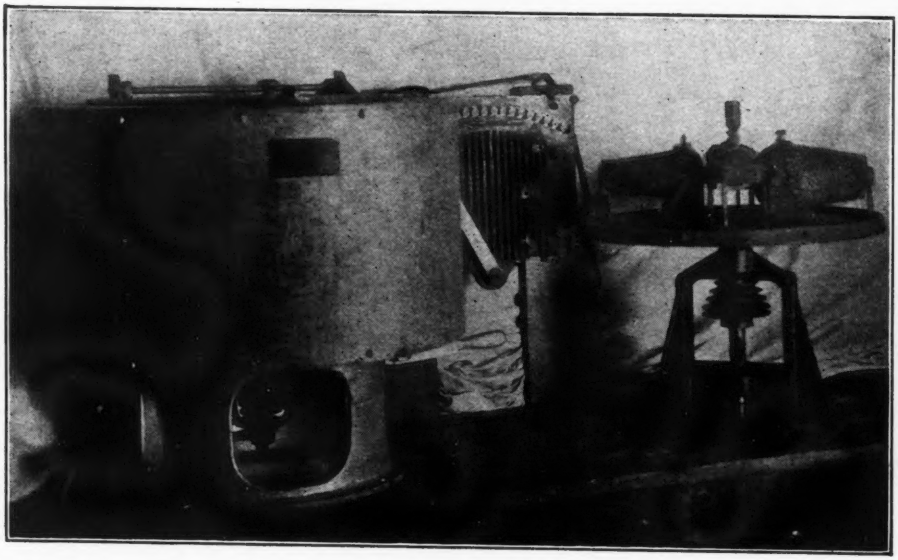


FIG. 3.—CENTRIFUGAL ELUTRIATOR (ON RIGHT), AND CENTRIFUGAL COMPACTOR (ON LEFT).

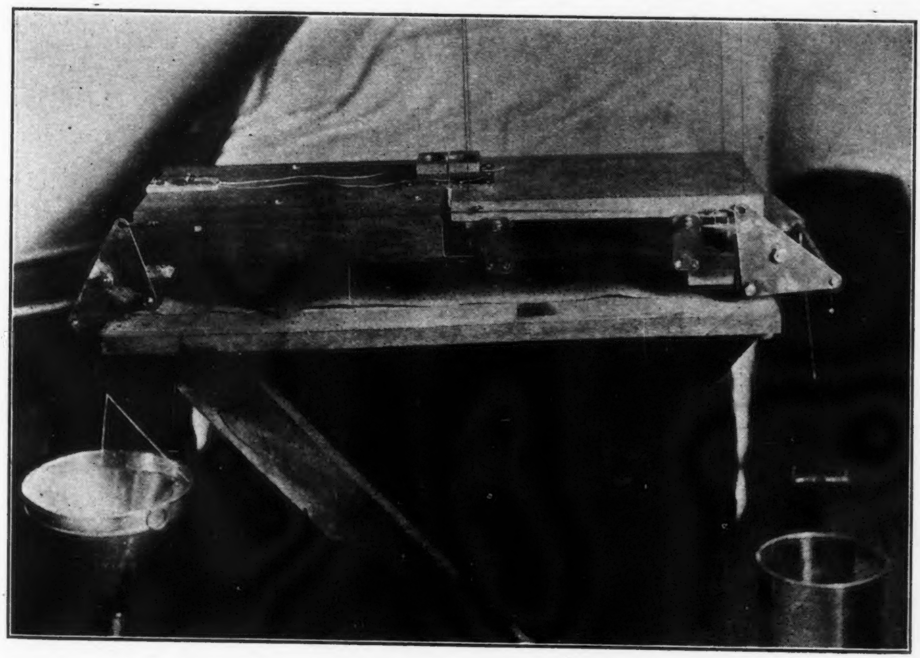
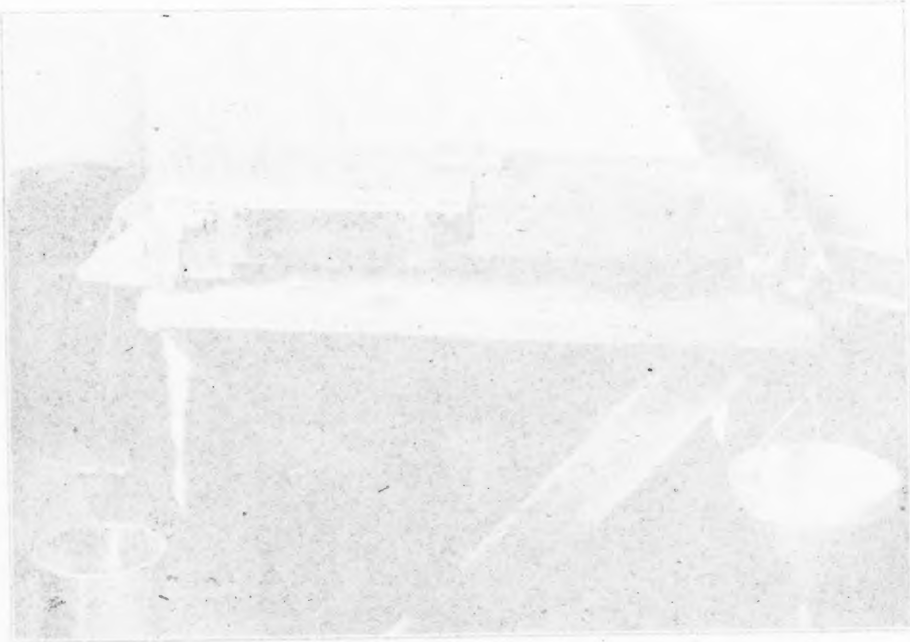


FIG. 4.—TENSION AND SHEARING PLATFORM.



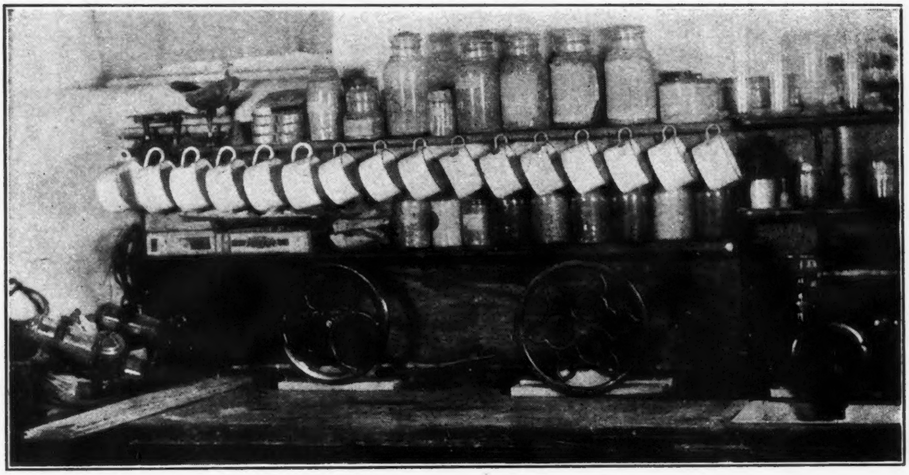


FIG. 5.—SAMPLE WASHING MACHINE RUN BY WATER MOTOR.

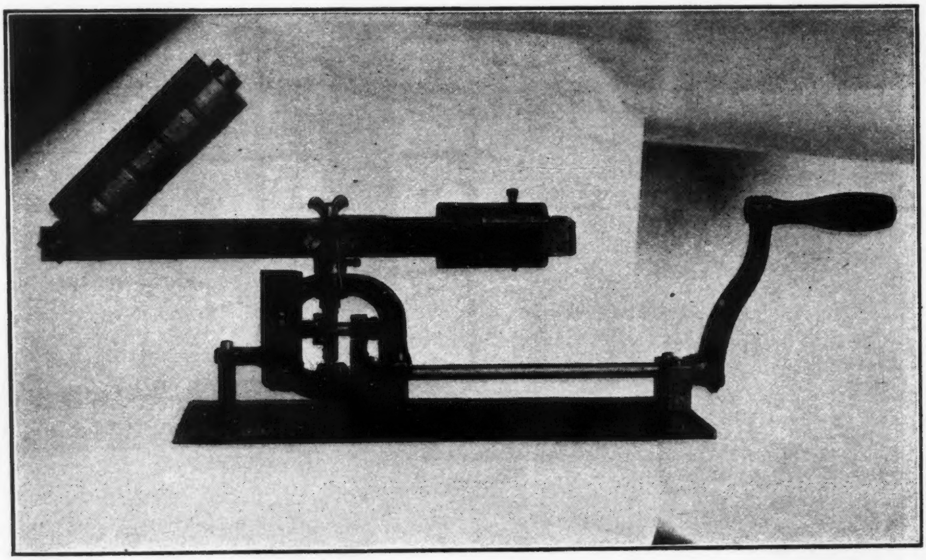


FIG. 6.—CENTRIFUGAL SCREENING APPARATUS.

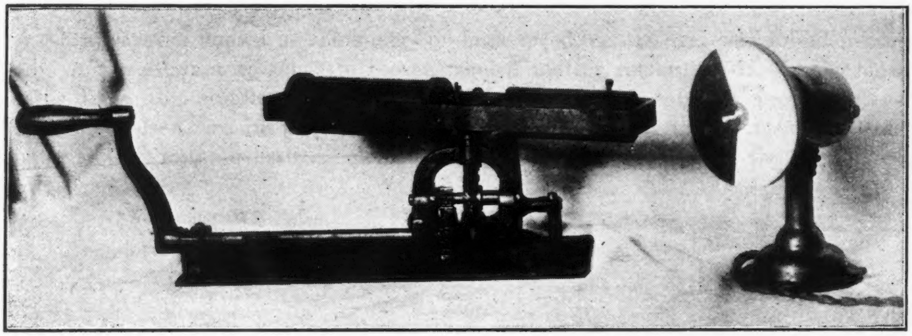


FIG. 7.—SCREENING APPARATUS (LEFT), AND ROTATING COLOR DISK (RIGHT).

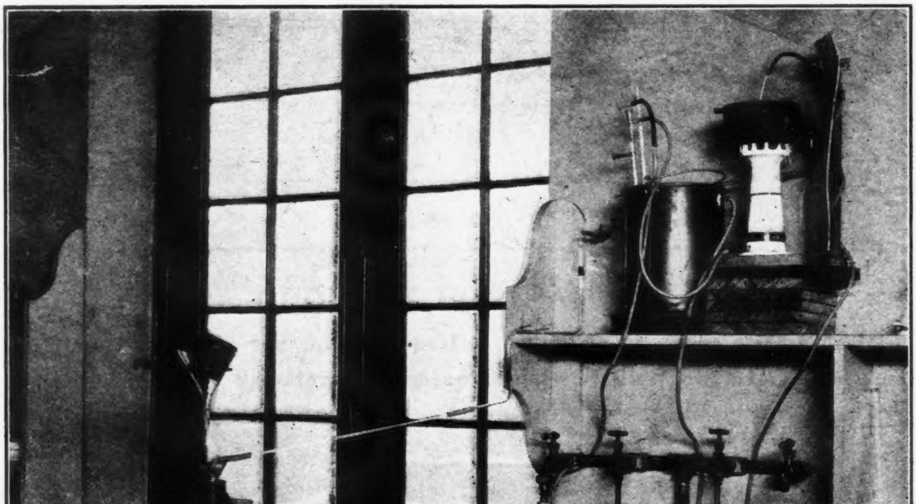


FIG. 8.—SCHULTZE ELUTRIATOR.

An apparatus known as the super- or high-speed, centrifuge, was found necessary in the attempt to separate the suspended matter remaining in the overflow water from the elutriators. This material has marked cohesive properties, as mentioned elsewhere in the report. All indications point to the fact that this material and water constitute elements of extreme importance in the physics of soils.

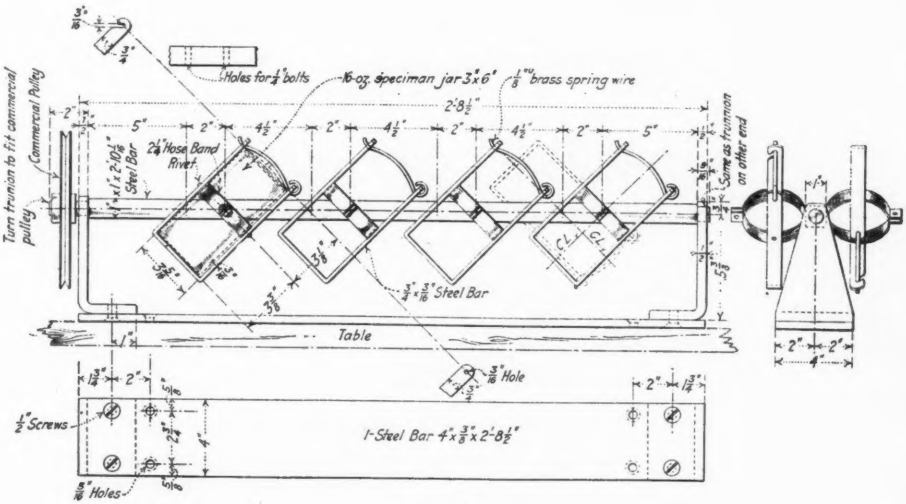


FIG. 9.

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SYNOPSES OF PAPERS

THE FLOW OF LIQUIDS THROUGH SHORT TUBES*

BY WINSLOW H. HERSCHEL, ESQ.†

SYNOPSIS.

In hydraulics a short tube usually receives scant attention, but it has two extensive uses. The carburetor nozzle is a short tube which limits the supply of gasoline, and with most viscosimeters the time of flow through a short outlet tube is used as a measure of viscosity. It is proposed from an analysis of published data in regard to long tubes and orifices, supplemented by the small amount of data available on short tubes and by original tests, to study the laws of flow through short, smooth tubes.

It is believed that the paper will prove to be of interest to hydraulic engineers.

* This paper will not be presented at any meeting of the Society, but written communications on the subject are invited for distribution and publication with the paper in *Transactions*. Published by permission of the Director of the U. S. Bureau of Standards.

† Associate Physicist, U. S. Bureau of Standards, Washington, D. C.

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It contains, in addition to a summary of the literature of the subject, the following original features:

1.—Unpublished tests on an "inverted" Saybolt Universal viscosimeter.

2.—The determination of the kinetic energy correction for turbulent flow from the experiments of Couette. The value is not $\frac{v^2}{2\,g}$, as ordinarily assumed in hydraulics.

3.—The determination of the length of tube having the same law of flow in the stream line and in the turbulent régimes.

4.—Proof that Sorkau was in error in thinking his tests showed a lower value of Reynolds' criterion than that usually determined at the critical velocity.

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SYNOPSES OF PAPERS

A STUDY OF STREAM FLOW: A COMPARISON BETWEEN THE FLOW AS OBSERVED AT TWO SEPARATE POINTS ON THE KERN RIVER, CALIFORNIA*

By H. W. Dennis,† M. Am. Soc. C. E.

Synopsis.

In this paper the writer undertakes to show the results of a rather extended study of the flow of the Kern River in the State of California, wherein it was desired to make use of a long period of observations of stream flow taken on this

* This paper will not be presented at any meeting of the Society, but written communications on the subject are invited for distribution and publication with the paper in *Transactions*.

† Los Angeles, Cal.

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river at a point where the water is diverted for irrigation purposes, and to apply these records to a point about 75 miles distant on the same river where it was contemplated to divert the water for power purposes. Before the power development could proceed, and, in fact, before its desirability could definitely be established, it was necessary to establish the probable performance of the plant, not only for the proper design of the structures, but for the determination of the probable output in relation to the anticipated cost. This is the same problem always encountered when any development of magnitude is contemplated, either in hydro-electric power or in irrigation, for in any such anticipated work the two things which must be predicted are: First, the cost of the work, and, second, its performance.

The writer undertakes to show that the Kern River, which is a typical Southern California stream, has entirely different characteristics during different periods of the year, and that the direct comparison of stream flow between two points must take into consideration the variation in stream characteristics throughout the year, or else the results obtained will lead to erroneous conclusions. He also undertakes to show that without consideration of these variable characteristics the resulting computations of low-water run-off, during the period when the river is not supported by storm, will be too high, and that the conclusions as to run-off during the period when the stream is supported by the water from melting snow will be too small. If these conclusions are substantiated, he will show that the results from the application of ordinary methods will be misleading not only as to the question of the cost of work and the design of the structures, but as to the performance on completion.

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ADDRESS

AT THE ANNUAL CONVENTION AT HOUSTON, TEX., APRIL 27TH, 1921.

MUNICIPAL ENGINEERING

BY GEORGE S. WEBSTER, PRESIDENT, AM. SOC. C. E.

In the performance of my duty as President to deliver an address at the Annual Convention I have selected the subject "Municipal Engineering", in which work I have been engaged during all my professional career. While my remarks may in some measure reflect conditions as I have observed them during my connection with municipal management in the City of Philadelphia for more than four decades, I believe these conditions are largely typical of those in other American cities.

The term "Municipal Engineering" did not come into general use until the latter part of the Nineteenth Century; it is that branch of Civil Engineering especially related to the problems of municipal corporations, and includes the planning, construction, and operation of public improvements and utilities required for city growth and development, for furnishing the citizens and industries with certain commodities needed for health, commerce, and prosperity, and for removing and disposing of the wastes which are detrimental to health and to the well being of the people.

ORIGIN OF MUNICIPAL ENGINEERING.

The rapid increase in urban population, with the consequent problems involved, has necessitated making adequate provision for the public service required for the social, business, and industrial enterprises of these people and for their comfort, convenience, and safety, and has been the chief agency in developing Municipal Engineering and raising it to a position of importance equal to that of other branches of engineering.

In the days of the small city, as transportation, sanitation, water supplies, and other public works first became essential, great waste often existed in public improvements by reason of the fact that there was an absence of a comprehensive understanding of the problems and of their proper solution. Important public

works were often in charge of men selected without regard to their training and qualifications for the work. Hence, the development and operation of cities were not always conducted efficiently.

As cities grew, and the demand for improved streets, sanitary drainage, water supplies, and other public service increased, greater engineering skill was required to meet these demands, until at this time, cities have become such tremendous working organisms, and their problems of physical development so varied and complex that the chief engineer of the modern city, to meet his responsibilities, must have skill in a wide field of engineering. He must also have foresight and initiative in discerning and anticipating the evolution of engineering enterprises as they affect the progress and prosperity of the city. Although most of the work connected with the growth and development of cities comes under the province of Civil Engineering, yet the increasing importance of specialization has developed the distinctive branch of Municipal Engineering.

South Philadelphia Improvement.

In Philadelphia, just prior to the breaking out of the World War, there was organized, officially authorized, and financed, the most comprehensive city-planning project ever actually undertaken as a single co-ordinated enterprise in the United States. This is known as the South Philadelphia Improvement, and affected an area of more than 8 sq. miles. It provided (1) for a revision and extension of the street system in accordance with the modern theory and practice of city planning; (2) for such a change in the methods of land subdivision as would insure better homes for the people than it had been customary to build in that section of the city; (3) for a liberal apportionment and systematic distribution of parks, parkways, playgrounds, public squares, and other open spaces, and for local, civic or neighborhood centers; (4) for industrial and commercial areas; (5) for a complete drainage system of an unusually complex nature; (6) for the complete relocation and reorganization of a network of main and branch railroad lines and freight yards belonging to two great railroad systems and their reconstruction under a joint traffic agreement permitting their use by foreign companies; (7) for the abolishment of grade crossings, the construction of elevated railroads and railroad and highway bridges; (8) for the construction of terminal yards and piers for the transfer of rail and ocean freights; (9) for harbor improvements; (10) for the development of wide marginal ways along the harbor for commercial uses; and (11) for the acquisition of large areas of harbor frontage and the construction of an extensive system of municipally owned and operated piers.

The work authorized and commenced under ordinances, contracts, and agreements of the city and the railroad companies, called for an expenditure of more than \$25 000 000, all of which was pledged and available. Had the war not intervened, this work, which was little more than the beginning of a great project of comprehensive city development, would now be completed.

All the preliminary investigations and studies and all the plans, estimates, contracts, ordinances, and agreements involved in the conception, promotion, and execution of this many-sided project, were prepared by the engineering staff of the city, except such as related wholly to the construction of railroad lines, yards,

and piers, although when such construction affected the public or city's interests it was subject to inspection and approval by the municipal engineers.

The completion of this project has been deferred but not abandoned and will undoubtedly go forward as soon as the railroad companies are again on a safe financial footing. It is referred to here as being perhaps the best practical illustration that can be given of the scope and range of Municipal Engineering in a great city and of the obligations and responsibilities of the Municipal Engineer.

Growth and Development of Municipal Engineering.

To consider properly the growth and development of Municipal Engineering it is necessary to consider them under the following divisions.

First.—The functions which have to do with the development and growth of cities:

a.—City planning.
b.—Transportation.
c.—Port development.
d.—Bridges.
e.—Street pavements.

Second.—The means for bringing to the citizens the publicly furnished commodities:

f.—Water supply.
g.—Gas and electricity.

Third.—The means for removing wastes from the city:

h.—Sewerage.
i.—Refuse disposal.

City Planning.

During the past quarter of a century the evolution of cities and the revelation of the disastrous results that have sprung from the lack of intelligent control and co-ordination of the elements of urban growth have brought forward a new constructive activity, namely, city planning. Although this new activity calls to its service the skill and genius of all the professions, arts, and sciences which can contribute to city building, its ultimate success, like the ultimate success of any great forward step in constructive progress, depends on the co-operation of the engineer, and particularly of the Municipal Engineer, with his knowledge of the present and future needs of the city. Through his daily contact with its problems, he knows what standards of public works and service are necessary for the welfare of the people and the best expression of community life.

Fundamentally, city planning has to do with the laying of the foundation of the city, on the stability of which the usefulness of the structure raised depends. The failure of the city structure based on a small-town foundation to carry the service load of the modern city has been the real driving force behind the city-planning movement of the past twenty-five years.

The street system is the basis of every city plan and is the controlling element in determining the efficiency and economy of public works; its great importance lies in the fact that it provides channels for traffic, facilitates the subdivision and use of land, and creates opportunities for the expression of civic art. It is

the most difficult and costly part of the city's structure to change after it has been fully improved.

Most of the cities of America began their existence with an orderly layout of streets over an area of such extent as the founder or founders considered necessary for the enterprise they had in mind; when the growth reached the limits of this orderly layout it was almost invariably extended into new areas without any regard for order, system, or topography. Cities have usually grown by building up land subdivision after land subdivision on private initiative, each independent of all the others, with streets considered only in their effect on the salability of lots and without regard for their traffic value.

Until within a comparatively recent period it was customary in the initial layout of a town site to establish a severely rectangular system of streets regardless of topography and to adopt arbitrary and uniform standards for street widths and block dimensions. This custom has imposed on cities which have grown to any considerable size, very complex and costly problems of replanning and reconstruction which the Municipal Engineer is called on to solve. Transportation and drainage—both engineering problems of the greatest importance in economic city development—were virtually overlooked to become later the subjects of controversy and large expenditures. Much of this could have been avoided by the exercise of engineering skill and foresight in laying out the streets in such a manner as to permit of transportation lines and drainage channels being constructed along direct lines of least resistance.

An economic street system calls for such an arrangement of its units and such a differentiation of their widths as will reduce to a minimum the amount of land used for such purposes and still provide sufficient street areas in proper locations to care adequately for all the services of every character which may be placed in or on or above them. This calls for a most thorough and forward-looking survey by the Municipal Engineer of the probable nature and extent of the city's growth, and the character and amount of service required of the streets to meet the needs of that growth. The street system, or at least those streets which are to serve as main thoroughfares or arteries of city circulation, should be planned for in advance of other urban improvements and in a manner to meet all future requirements.

City Transportation.

The traffic on the highways of our cities has grown so rapidly in the past few years, especially since the advent of the automobile and motor truck, as to become a serious problem, claiming the earnest attention of the authorities. As our cities continue to grow, and as trade and business continues to be increasingly concentrated at one or more centers, the problem of transportation of all kinds through the city becomes more difficult. The economical administration of industry and the transaction of business in cities are dependent on the ease with which materials and merchandise may be moved, for modern business requires that transportation shall be by the most direct route and in the shortest time.

Inseparably interwoven into the street system and its functions are the facilities of transportation, whether they relate to the local movement within the city itself or to that wider movement through which the city maintains its contact with the world. Transportation is essentially a problem of planning

constructive and administrative engineering with which the municipal engineer must keep in touch if his plans for city development are to function smoothly. Rapid and convenient urban and interurban service depends on the extent to which the system of main traffic streets has been planned to connect the important city centers by the most direct and adequate routes. The vigilance and persistence of the municipal engineer is necessary in securing the proper adjustments of both street and railroad grades and in preventing or avoiding the cutting off or abandonment of important streets in the construction of railroads and yards. In devising plans for getting rid of the grade crossings which are still a menace to life and property and an obstruction to traffic in many of our cities, he is the most important representative of the public.

Port Development.

Nearly all the large cities of this country are located on navigable waterways—many of them being situate on deep estuaries leading direct to the ocean. The World War has resulted in the creation of a great international trade between this and foreign countries. To maintain this trade successfully in competition with other countries, it is necessary that the most modern facilities for handling and shipping goods shall be provided. Port authorities in every city on the Atlantic, Pacific, and Gulf Coasts, and on our Great Lakes, anxious to share in this foreign trade, have been actively engaged during the past few years in developing their terminal facilities and are now planning greater extensions to handle the additional water-borne cargoes. In order that a port may compete in the world trade it is essential that provision shall be made in the planning of the city for the great trunk railroads to reach the water-front, either directly or over a belt line railroad system, so that the cars may deliver cargo at the ship's side. It is also necessary that a system of traffic streets shall be laid out and developed in the rear of the piers and along the water-front, to give highway facilities for motor trucks and vehicles to make deliveries to and from the industries, warehouses, and stores located in the vicinity of the water terminal.

The authority to plan and administer the ports of this country is vested generally in the officials of the city, although there are several instances where ports are under the control of State commissions; but in all cases the development of the land side of the port is a proper task for men skilled in both city planning and other municipal work.

City Bridges.

No construction work attracts more attention and receives more comment from the public than the bridges which the city engineer designs and constructs. In no other branch of the Profession has there been greater progress, especially along esthetic lines, than in the art of bridge building. It was formerly the usual practice when an important bridge was to be erected to specify the needs in the way of travel and loading and to ask contractors and bridge-building companies to submit bids with plans and specifications for the type of structure they proposed to erect for the prices bid. This method did not always prove satisfactory to either party, and was found to be uneconomical. Now, in most of the large cities, the engineering forces of the municipality prepare plans for the bridge to be erected, giving every detail of construction, and specifications defining the

quality of the materials to be incorporated in the work, as well as the character of the workmanship; this results in real competition and economy in construction. The same department supervises the construction and is generally charged with the maintenance.

The utilization of concrete and reinforced concrete in recent years marks an epoch in the art of bridge building and has resulted in great improvement in the architectural features and appearance of bridges. Artistic cornices, projections, balustrades, and other ornamentation can be obtained with concrete at little expense. This fact makes concrete a valuable and desirable material for use in building bridges in public parks, in suburban and residential sections, and along parkways and boulevards. Engineers were formerly content to build bridges for their strict utilitarian use, but at present, particularly since the advent of concrete, greater attention is being given to the architectural features.

Street Pavements.

Fifty years ago the paving and maintenance of the highways in many cities frequently were in charge of men unskilled, and selected by political preferment. Now, due largely to the activities of civic and business organizations interested in street betterments, and also to the advancement in municipal administration, work of this kind is usually entrusted to trained engineers familiar with municipal affairs.

The development and increase of motor traffic in cities has led to an improvement in its pavements to meet this demand. Materials heretofore found to be satisfactory have proven to be inadequate, and this has necessitated the development of road surfaces which will give maximum wear with a minimum cost of construction and maintenance. Laboratories have been established for research and to provide means for determining the properties of materials. Comprehensive specifications are now drawn in which the materials to be used are definitely described and the methods of tests to insure such materials are clearly set forth. This enables the city to obtain proper construction of its street surfaces and to effect great economies, due to the fuller and freer competition of bidders, and greater permanency of the work.

Water Supply.

Providing purified water supplies and comprehensive sewerage systems are the principal functions of Municipal Engineering which have a direct bearing on public health. As recently as the middle of the Nineteenth Century, these two great questions were not always considered as engineering problems.

Private supplies of water were obtained ofttimes from dug wells in close proximity to the dwelling-house; excreta were disposed of in privy vaults and cesspools which, through underground or surface overflow, made possible the contamination of the near-by water well.

It had long been the custom to construct rough masonry culverts to enclose the streams flowing through the town, and it gradually became the practice to permit overflows from cesspools and also connections from water-closets to be made to these crude storm-water drains, thus conveying the sewage to a watercourse from which a public water supply was taken without even a thought

of purification. As the cause of typhoid fever and other water-borne diseases was not known at that time, little heed was paid to such contamination.

Water Filtration.

Filters for the improvement of public water supplies were first used in Europe, and, about 1866, the late J. P. Kirkwood, Past-President, Am. Soc. C. E., went abroad for the purpose of examining such works. Based on the data he obtained, municipal water filters were constructed and successfully operated at Poughkeepsie, N. Y. The Massachusetts State Board of Health, in 1890, began experiments on the filtration of water, and shortly afterward municipal water filters were designed for Lawrence, Mass., based on these experiments. The filtration of the Merrimac River water at Lawrence demonstrated the beneficial effects on the public health of purifying water by filtration, not only as evidenced by the reduction in typhoid fever, but also by the lowering of the total death rate.

When the City of Louisville, Ky., conducted experiments in 1895 on the filtration of the Ohio River water, it was found that the practice of sedimentation and slow sand filtration known at that time was not applicable to a raw river water containing large quantities of very finely divided suspended matter. In these experiments there was developed the chemical coagulation of the raw water and its subsequent mechanical or rapid filtration now so commonly used.

About 1908 the sterilization of public water supplies with a solution of calcium hypochlorite was begun and has developed very rapidly. Later, chlorine gas was used directly for the same purpose.

The control of sparsely inhabited water-sheds, which minimizes the danger of contamination, is resorted to for the purpose of avoiding artificial purification at the points of consumption. Generally, even such water is also sterilized before delivery to the consumer.

The comparative cheapness of unfiltered water creates in the minds of people the idea that water should be "free as air", and hence there results great waste in its use. The Municipal Engineer recognizes that artificially purified water is really a manufactured product, and economy of public funds demands curtailment of waste; therefore the water meter was introduced to provide a measure for the water actually used, for which payment can be exacted.

Gas and Electricity.

The manufacture and distribution of gas and electricity and the lighting of the public streets are distinctly municipal problems. Whether the city owns and operates the plant or whether it purchases the commodity from a utility company, the services of a specialist are required, either to administer or to supervise the operations, and to see that standards are maintained.

Sewerage Systems.

It is only within the past few decades that the public has come to a full realization that the health of the community very largely depends on a properly constructed and maintained sewerage system by which the liquid wastes of the community are promptly removed from their place of origin. Sewerage systems had their origin in America, as in Europe, in the construction of masonry culverts

to carry the small streams through the built-up portions of the city. There appears to have been little or no actual design of such culverts based on hydraulic principles, and when lateral extensions were made for conveying rain-water underground to the main culvert, the size of the lateral was not determined by the quantity of water to be carried, but rather by the diameter required for workmen to enter the culvert to remove deposits formed therein on account of insufficient velocities. In some cities the size of lateral sewers was arbitrarily fixed at 3 ft. in diameter.

In 1842, Lindley, an English engineer, laid out the sewerage system of Hamburg, and in his design provided for the continuous movement of the sewage by sufficient gradients and with periodic flushings.

Design of Sewers.

The design of American municipal sewerage systems on a comprehensive plan rather than piecemeal construction, was begun in 1855 in Chicago, Ill., by the late E. S. Chesbrough, Past-President, Am. Soc. C. E., followed shortly in Brooklyn by the late J. W. Adams, Past-President, Am. Soc. C. E., and in Boston by the late J. P. Davis, M. Am. Soc. C. E. These designs made use of empirical data obtained from European practice as to capacity and as to probable quantities of rain-water to be carried by the sewers.

The sewers of this early period, being developed from the old culverts, naturally were on the combined system, providing for the carriage of both sewage and rain-water in the same conduit. In England, as early as 1842, Chadwick had advocated the separate collection of sewage and storm-water, but it was not until 1880 that such a system was installed in a large city when the late Col. Waring designed the separate system of sewers for Memphis, Tenn.

The then National Board of Health sent Rudolph Hering, M. Am. Soc. C. E., to Europe to investigate sewerage practice, and his report thereon marks one of the turning points in sewer design in this country, as it placed such design on a scientific basis rather than rule-of-thumb. About the same time, the Burkli-Ziegler formula for determining the rate of run-off of rain-water was published and marked an advance in design, as it took into consideration the acreage and territorial slope of the drainage area tributary to the sewers. Subsequent modifications of this formula were devised for a number of American cities in efforts to obtain more accurate results.

Sounder principles of structural design were then introduced, proportioning the various parts of the sewer to the probable loads they would be required to carry and securing smoother interior surfaces.

What is known as the rational method of determining the maximum rate of run-off of storm-water is now quite generally used by competent designers, and many large cities maintain gauges for recording the rate of rainfall and the rate of run-off in the sewers. These data from existing sewers furnish the basis for future design.

The extension of the sewerage system of a city is now really necessary to its normal development. Modern living demands water supply and toilet conveniences in dwellings, and this requires facilities for the prompt removal of the sewage.

The broad problem of maintaining streams free from nuisance and in such a condition that they are suitable sources for public water supplies after purification, or for use in industry, is a State function, but the maintenance in a clean condition of streams flowing through or by a city is a municipal problem.

Development of Methods of Sewage Disposal.

Before considering the evolution of sewage disposal in America, it is advisable to note briefly the development of the art in England where the necessity for sewage treatment arose earlier than in other countries by reason of the dense population, numerous industries, and the relative smallness of the streams.

About the middle of the Nineteenth Century, in order to clean up the English towns, it became common practice first to utilize the existing storm-drains and, later, to build new sewers for the conveyance of filth to the near-by water-courses, which naturally resulted in the serious pollution of the streams. Parliament, therefore, in 1857, created the Royal Sewage Commission, which, in its final report made in 1865, recommended the abatement of stream pollution by the application of sewage to land, that is, irrigation. Subsequently, the Second Royal Commission on Rivers Pollution, appointed in 1868, made its report in 1870, and recognized irrigation, intermittent filtration, and chemical precipitation as the then available processes of sewage treatment.

Meanwhile, many of the streams of Massachusetts had become polluted by the discharge into them of sewage and industrial wastes, and, in 1875, the State Board of Health made examinations of some of these rivers. Largely based on European practice, and bearing in mind the glacial formation common in Massachusetts, the Board recommended to inland towns the disposal of sewage by irrigation.

Fifty years ago the engineer in America had little choice of methods to prevent pollution of streams by city sewage. Worcester, about 1887, decided to use chemical precipitation to prevent the pollution of the Blackstone River caused by the discharge of crude sewage, and in the same year the Massachusetts State Board of Health began the now classic series of experiments on purification of sewage by intermittent filtration through beds of sand.

The effluent produced was clear, perfectly stable, and ofttimes of lower bacterial content than many well waters, and hence there arose in the minds of American engineers and sanitarians of that period the idea that "sewage disposal" meant the conversion of sewage into an effluent almost of drinking-water purity, and they utterly disregarded any subsequent purification in the receiving body of water through natural agencies.

The enormous quantities of watery, offensive sludge produced by the chemical precipitation of sewage, caused engineers to search for a process lacking this serious characteristic. There resulted the development of the contact bed in 1891 by the engineers of the London County Council, and thereafter this process became quite popular. Two years later, Corbett, at Salford, England, devised the trickling filter, which permitted higher rates of application than the contact bed and is to-day the most intensive, well established process for the oxidation of settled sewage.

The "activated sludge" process of sewage treatment was experimentally developed at Manchester in 1914. On this principle several installations have been made subsequently in both England and America.

Sedimentation Tanks.

Simultaneously with the development in processes for oxidizing sewage, there was a corresponding evolution in the means for freeing the crude sewage of its suspended matter as a preliminary process. For many years it had been known that the organic solids in sewage were susceptible to decomposition and liquification, but it was not until 1895 that Cameron, at Exeter, England, utilized these ideas on a large scale in a sedimentation tank intended to retain for a long time the deposited sludge.

The publication of the results accomplished in the Exeter septic tank led to the belief that at last a process had been found which practically eliminated the sludge problem that was such a serious one in chemical precipitation, and the result was that municipal engineers everywhere installed septic tanks, fondly trusting that they would be the remedy for their troubles. Extensive experience soon showed that the deposits were not all "digested", so that it was necessary to remove the sludge from these tanks, and that the decomposition of the sludge impregnated the effluent of the tank with offensive gases.

In 1904, Travis, at Hampton, attempted to overcome the latter difficulty by constructing a false bottom in the tank, which permitted about one-fifth of the raw sewage to flow through the lower sludge compartment. This resulted in a slight improvement in the quality of the effluent, but still produced offensive sludge. The complete separation of the settling sewage from the digesting sludge was devised by Imhoff about 1906, and tanks were constructed on this principle. The results of their operation were published in an English sanitary engineering paper in May, 1909, and in the following July an experimental tank on this principle, was put into operation by the City of Philadelphia in the Spring Garden Sewage Testing Station.

The intense interest and enthusiasm shown for this type of tank seemed for a time likely to reproduce the previous experience with the septic tank. Although some improperly designed tanks have not been successful, and other well-designed tanks have caused trouble, particularly in their early operation, due to foaming from gas vents, it appears to-day, particularly for large installations, to be the safest sedimentation tank, both from the point of view of removal of settleable material and production of minimum volumes of least offensive sludge.

There is a tendency to-day to accomplish the same results through the frequent removal of sludge from a plain sedimentation tank to a separate tank for subsequent digestion.

The application of a solution of calcium hypochlorite as a chemical germicide to the effluent of a municipal sewage works was first made in 1907 by Daniels at Red Bank, N. J., and has become common practice where the destruction of pathogenic bacteria is deemed necessary to protect sources of public water supplies or shell-fish beds.

Since 1877, it has been known that the chemical changes which sewage undergoes in its artificial treatment are produced through living organisms, but the

utilization of these same biological forces in natural waters has only been scientifically considered within recent years.

Notwithstanding the one-time almost universal practice of discharging raw sewage into streams, disposal by dilution was not considered as a method of treatment. The first attempt of an American municipality to dispose of sewage by dilution, as an avowed process for its purification, was made in the construction of the Chicago Drainage Canal. From that date there has been a gradual accumulation of knowledge as to the conditions which are necessary to provide for the inoffensive assimilation and complete oxidation of sewage through properly distributing it so as to bring about good diffusion throughout the cross-section of the receiving body of water. To-day, large sums of public money can often be saved by utilizing these natural powers in streams where conditions are such that the discharge of untreated or partly treated sewage will not constitute a menace to the public health.

Refuse Disposal.

The problem of the sanitary collection and disposal of the solid refuse of the city has been only recently taken up for scientific investigation by the Engineering Profession. Prior to that time all work of this kind was left to the householder and the man who collected garbage to feed to hogs. These methods were irregular and not dependable. Nuisance and serious inconvenience resulted, and the city authorities have been compelled, as a health measure, to assume the responsibility and make this work a part of the duty of the municipality.

The gathering of the city's wastes and its economical disposal or destruction is one of the latest but not least important problems of Municipal Engineering, for on its proper carrying out depends the health and comfort of the people.

Opportunity for Service.

As I review my experience of many years covering my connection in the practice of Municipal Engineering, I more fully appreciate the changes and improvements which have taken place in city government, whereby efficient and adequate public service has been provided to care for the rapidly increasing demands of the community. Our cities are more scientifically planned, have improved methods of transportation, are better paved and lighted, are provided with a more abundant supply of pure water, and the disposal of waste is properly cared for.

The Municipal Engineer has probably greater opportunity for service than those in other branches of the Profession, since he has to do with those problems which deal directly with the welfare of the people. If he uses his training and skill tactfully and wisely he will gain the confidence of the community and thus become a factor for good. By exercising foresight and initiative, he should be the leader in anticipating and directing the affairs and the development of the community.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

SYNOPSES OF PAPERS

VERTICAL LIFT BRIDGES

BY ERNEST E. HOWARD,* M. AM. SOC. C. E.

PRESENTED MAY 4TH, 1921.

SYNOPSIS.

The purpose of this paper is to outline the development of the vertical lift bridge, to discuss its principal elements, and to describe noteworthy features of some structures in which it is used. Part I deals with the general elements of lift-bridge design; Part II describes a bridge with a lifting span, the Columbia River Interstate Bridge; Part III describes a bridge with a lifting deck, the North Kansas City Bridge; and Part IV describes a bridge with a lifting span which has a lifting deck, the Harriman Bridge over the Willamette River at Portland, Ore.

Twenty-five years ago the swing span was almost the only type of movable span in use, but its limitations and deficiencies brought engineers to devise spans which would move vertically instead of horizontally. These include many ingenious bascule bridges which rotate in vertical planes, and the lift bridge which moves vertically. Although a rare type twelve years ago, more than forty lift spans have since been built, many in large and important structures. Since only fragmentary information about these structures has hitherto been available, the writer undertakes to describe in some detail the essential features of the foregoing bridges, as containing elements which typify the development of the lift-bridge idea, and as showing adaptability to widely varying conditions and limitations.

The paper indicates the advantages of the lift bridge—economy in construction, rapid operation at low cost, determinate stresses, and no new stress conditions introduced during operation. It may within reason be of any length, any width, of any material, and with any type of floor and pavement. Future alterations in the lifting span, or its floor, and future grade changes are readily allowed for; and the effect of wind on operating machinery is very slight. It is shown that few engineering elements are more reliable, efficient, and widely used than wire ropes, and most important lift spans are suspended by such ropes, although built-up chains, wrought chains, systems of levers, etc., have been considered and even tried. For counterweights for lift bridges cast iron has been used, but concrete built around a steel framework is commonly used as it costs less than one-third as much as cast iron.

* Kansas City, Mo.

The paper illustrates the operating machinery—ordinary hoisting equipment, simple and easily adjusted, and indicates that simple spur gearing ordinarily suffices, the longest shaft extending transversely from truss to truss. Electric or hand brakes, limit switches for automatic emergency stops, gasoline engines or electric motors, suitable position indicators, etc., are used.

Direct comparison of the operation of various types of movable spans in which the lift span is found to operate more quickly and costs less per operation than adjacent swing and bascule bridges, is made, speed of operation being due to the fact that the lift span need only be lifted high enough to pass the approaching boat (often only 15 or 20 ft.), and that there are no wedges or locks to be driven after the span is seated.

The Columbia River Interstate Bridge described in Part II includes a simple lift span 275 ft. long, 50 ft. wide, with a concrete floor and sidewalk, which will lift to a clear height of 150 ft. Two others of the twenty-nine spans of the structure are arranged to be converted into lift spans by the erection of towers, counterweights, machinery, etc., should increased navigation require it. The somewhat unusual methods of pier construction, superstructure erection, pumping embankment approaches, floor details, low construction costs, and the public acquirement of the property without taxation are notable in this bridge.

It is shown that the lift-bridge idea may be applied not only to lifting spans, but also to a bridge floor supported at several points along its length, called a lifting deck, Part III, which is peculiarly adaptable to double-deck structures. In older forms, of which a few small spans have been built, the lifting deck and

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its counterweights were supported from the overhead trusses. The later structures, as described, relieve the supporting trusses of about half this load by placing the counterweights at the ends of the span. The North Kansas City Bridge over the Missouri River has a lifting deck, 425 ft. long, serving a double-track railway. The special problems presented by the conditions and satisfactorily solved by the lifting deck are described.

The Harriman Bridge over the Willamette River at Portland, Ore., described in Part IV, offers a noteworthy example of the exceeding feasible variations possible with the wire-rope lift-bridge idea, for it has a lifting deck suspended under a lifting span which can be operated without movement of the span; or both can be raised at will. This design met the special condition of dense highway traffic on an upper level which is obstructed only a few times a day for the passage of masted vessels, although the lower deck is open sometimes for as many as one hundred boats per day. The river having a maximum depth of 90 ft., the piers extend to depths exceeding 140 ft., involving unusual construction methods which are described.

Quantities and costs of these three special bridges are included, and Table 1 gives general data of the more important lift bridges that have been built.

Members who desire a copy of this paper in full are requested to fill out the order blank and forward it to the office of the Secretary. The paper contains 47 pages, including 4 tables, and is illustrated by 3 diagrams and 13 half-tones.

Vol. XLVII. AUGUST, 1921. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

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Vol. XLVII. AUGUST, 1921. No. 6.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

TENTATIVE SPECIFICATIONS FOR CONCRETE AND REINFORCED CONCRETE

SUBMITTED AS A PROGRESS REPORT OF THE JOINT COMMITTEE ON STANDARD SPECIFICATIONS FOR CONCRETE AND REINFORCED CONCRETE

AFFILIATED COMMITTEES

OF THE

American Society of Civil Engineers,
American Society for Testing Materials,
American Railway Engineering Association,
Portland Cement Association,
American Concrete Institute.

SUBMITTED TO CONSTITUENT ORGANIZATIONS,

JUNE 4TH, 1921.

PREFACE.

The Joint Committee on Standard Specifications for Concrete and Reinforced Concrete consists of five representatives from each of the following:

American Society of Civil Engineers,
American Society for Testing Materials,
American Railway Engineering Association,
American Concrete Institute,
Portland Cement Association.

This Committee is the successor of the Joint Committee on Concrete and Reinforced Concrete which was organized in Atlantic City, N. J., June 17th, 1904, and was formed by the union of special committees appointed in 1903 and 1904 by the above-named organizations, except the American Concrete Institute, which was added by invitation of the Joint Committee in 1915. The previous Committee presented progress reports in 1909 and 1912, and adopted a final report to its constituent organizations on July 1st, 1916.* It was the purpose of that Committee to prepare a Recommended Practice for Concrete and Reinforced Concrete. Its final report stated:

"The report is not a specification, but may be used as a basis for specifications."

The present Joint Committee is charged with the preparation of Specifications for Concrete and Reinforced Concrete, and in preparing these specifications is using as a basis the report of the former Joint Committee with such modifications as are necessary to make its recommendations agree with current practice, and such new data as mark advances in the art.

The initiative in bringing about the present Joint Committee was taken by the Committee on Reinforced Concrete of the American Society for Testing Materials on June 27th, 1917, when the Committee voted to request the Executive Committee of the Society to invite the Member Societies of the previous Joint Committee to co-operate in the formation of a new Joint Committee. The Executive Committee approved this request on April 25th, 1919, and an invitation was issued to each of the above-named organizations by the Executive Committee on behalf of the American Society for Testing Materials, to appoint five members on a Joint Committee on Specifications for Reinforced Concrete. The last of these organizations accepted the invitation on November 22d, 1919. On January 21st, 1920, a call for an organizing meeting on February 11th, 1920, was sent by the Executive Committee of that Society, to each of the twenty-five representatives of co-operating organizations, together with a list of members of the Joint Committee, and an outline of organization that had been previously submitted by the American Society for Testing Materials to, and approved by, the co-operating organizations.

The organizing meeting was held at the Engineers' Club, Philadelphia, Pa., and was called to order by George S. Webster, then Vice-President of the American Society for Testing Materials, who explained that he had been

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1101.

directed by the Executive Committee of that Society to act as Temporary Chairman; he further stated that C. L. Warwick, Secretary-Treasurer of the Society, had been requested to act as Temporary Secretary until a formal organization of the Joint Committee had been effected.

The personnel of the Joint Committee is as follows:

AMERICAN SOCIETY OF CIVIL ENGINEERS

RUDOLPH P. MILLER, *Chairman*, Consulting Engineer, New York City. Resigned March 28th, 1921. Succeeded as Chairman by

W. A. SLATER, Engineer-Physicist, Bureau of Standards, Washington, D. C.

WILLIAM K. HATT, Professor of Civil Engineering, Purdue University, Lafayette, Ind.

A. E. LINDAU, General Manager of Sales, Corrugated Bar Company, Buffalo, N. Y.

SANFORD E. THOMPSON, Consulting Engineer, Boston, Mass.

FRANKLIN R. MCMILLAN,* 628 Metropolitan Bank Building, Minneapolis Minn.

AMERICAN SOCIETY FOR TESTING MATERIALS

RICHARD L. HUMPHREY, *Chairman*, Consulting Engineer, Philadelphia, Pa

ALBERT T. GOLDBECK, Engineer of Tests, Bureau of Public Roads, Washington, D. C.

EDWARD E. HUGHES, Vice-President, Franklin Steel Works, Franklin, Pa.

HENRY H. QUIMBY, Chief Engineer, Department of City Transit, Philadelphia, Pa.

LEON S. MOISSEIFF, Consulting Engineer, New York City.

AMERICAN RAILWAY ENGINEERING ASSOCIATION

J. J. YATES, *Chairman*, Bridge Engineer, Central Railroad of New Jersey, Jersey City, N. J.

GEORGE E. BOYD, Division Engineer, Delaware, Lackawanna, and Western Railroad Company, Buffalo, N. Y.

FREDERICK E. SCHALL, Bridge Engineer, Lehigh Valley Railroad Company, Bethlehem, Pa.

H. T. WELTY, Engineer of Structures, New York Central Railroad, New York City.

C. C. WESTFALL, Engineer of Bridges, Illinois Central Railroad Company, Chicago, Ill.

AMERICAN CONCRETE INSTITUTE

S. C. HOLLISTER, *Chairman*, Consulting Engineer, Philadelphia, Pa.

ROBERT W. LESLEY, Past-President, Association of American Portland Cement Manufacturers, Philadelphia, Pa.

ARTHUR R. LORD, President, Lord Engineering Company, Chicago, Ill.

EGBERT J. MOORE, Vice-President, Turner Construction Company, New York City.

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* Appointed to fill vacancy.

LEONARD C. WASON, President, Aberthaw Construction Company, Boston, Mass. Resigned October 19th, 1920. Succeeded by

ANGUS B. MACMILLAN, Chief Engineer, Aberthaw Construction Company, Boston, Mass.

PORTLAND CEMENT ASSOCIATION.

FREDERICK W. KELLEY, *Chairman,* President, Helderberg Cement Company, Albany, N. Y.

DUFF A. ABRAMS, Professor in Charge, Structural Materials Research Laboratory, Lewis Institute, Chicago, Ill.

ERNEST ASHTON, Chemical Engineer, Lehigh Portland Cement Company, Allentown, Pa.

EDWARD D. BOYER, Cement Expert, Atlas Portland Cement Company, New York City.

J. H. LIBBERTON, Manager, Service Bureau, Universal Portland Cement Company, Chicago, Ill. Resigned January 1st, 1921. Succeeded by

J. E. FREEMAN, Manager, Structural Bureau, Portland Cement Association, Chicago, Ill.

The Committee perfected a permanent organization on February 11th, 1920, under the title "Joint Committee on Standard Specifications for Concrete and Reinforced Concrete" with the following officers:

Chairman, RICHARD L. HUMPHREY, Philadelphia, Pa.
Vice-Chairman, J. J. YATES, Jersey City, N. J.
Secretary-Treasurer, DUFF A. ABRAMS, Chicago, Ill.

and an Executive Committee consisting of these officers, and Rudolph P. Miller,* New York City, and S. C. Hollister, Philadelphia. Pa.

The Committee adopted Rules of Organization and apportioned the work of preparing a tentative draft of the specifications among sub-committees, the present personnel of which is as follows:

1.—*Materials (other than Reinforcing)*:
Albert T. Goldbeck, *Chairman,*
Duff A. Abrams,
J. E. Freeman,†
Sanford E. Thompson,
J. J. Yates.

2.—*Metal Reinforcement:*
J. J. Yates, *Chairman,*
Duff A. Abrams,
William K. Hatt,
Edward E. Hughes,
A. E. Lindau.

3.—*Proportioning and Mixing:*
W. A. Slater, *Chairman,*
Duff A. Abrams,
Ernest Ashton,
George E. Boyd,
Henry H. Quimby.

4.—*Forms and Placing:*
George E. Boyd, *Chairman,*
Edward D. Boyer,
Angus B. MacMillan,‡
Egbert J. Moore,
Frederick E. Schall.

* Succeeded by W. A. Slater, May 25th, 1921.
† Succeeded J. H. Libberton, January 1st, 1921.
‡ Succeeded Leonard C. Wason, October 19th, 1920.

5.—*Design:*
S. C. Hollister, *Chairman,*
William K. Hatt,
A. E. Lindau,
Arthur R. Lord,
Franklin R. McMillan
Egbert J. Moore,
W. A. Slater,
H. T. Welty.

6.—*Details of Construction and Fire-Proofing:*
Franklin R. McMillan,* *Chairman,*
William K. Hatt,
Arthur R. Lord,
Leon S. Moisseiff,
C. C. Westfall.

7.—*Water-Proofing and Protective Treatment:*
Frederick W. Kelley, *Chairman,*
Albert T. Goldbeck,
S. C. Hollister,
Robert W. Lesley,
C. C. Westfall.

8.—*Surface Finish:*
Henry H. Quimby, *Chairman,*
Edward D. Boyer,
J. E. Freeman,†
Angus B. MacMillan,‡
H. T. Welty.

9.—*Form of Specification:*
Richard L. Humphrey, *Chairman,*
Duff A. Abrams, *Secretary,*
George E. Boyd,
Albert T. Goldbeck,
S. C. Hollister,
Frederick W. Kelley,
Franklin R. McMillan,*
Henry H. Quimby,
W. A. Slater,
J. J. Yates.

The Joint Committee held the following meetings:

Organization meeting, Philadelphia, Pa., February 11th, 1920;
Second meeting, Asbury Park, N. J., June 23d and 24th, 1920;
Third " New York City, October 26th, 27th, and 28th, 1920;
Fourth " New York City, December 15th, 16th, and 17th, 1920;
Fifth " New York City, March 2d, 3d, and 4th, 1921;
Sixth " New York City, April 13th, 14th, and 15th, 1921.

At these meetings the Joint Committee considered the reports of its sub-committees, which were edited by the Sub-Committee on Form and incorporated in the Tentative Specifications for Concrete and Reinforced Concrete herewith submitted.

TENTATIVE REPORT IS SUBMITTED FOR CRITICISM AND DISCUSSION.

The Rules of Organization of the Joint Committee which were submitted to, and approved by, each of its constituent organizations, provide that,

"The initial report of the Joint Committee shall be considered by each of the five organizations as a tentative report submitted for criticism and discussion, limited to not less than six months nor more than one year. Such discussions shall then be referred to the Joint Committee for consideration in revising its report." (Article IX, Section 2.)

* Succeeded Rudolph P. Miller, May 25th, 1921.
† Succeeded J. H. Libberton, January 1st, 1921.
‡ Succeeded Leonard C. Wason, October 19th, 1920.

The Joint Committee, in submitting these Tentative Specifications for Concrete and Reinforced Concrete in accordance with the foregoing requirement, wishes it clearly understood that it reserves the right to make such changes as may be found desirable, after a further study of the available data. While not prepared to submit a final report at this time, the Joint Committee is of the opinion that the specifications are in such shape as to make it desirable to issue them tentatively for the purpose of facilitating the final submission of Standard Specifications for Concrete and Reinforced Concrete.

The Joint Committee earnestly requests that every facility be provided by its constituent organizations for the fullest consideration of these Tentative Specifications in order that it may be in a position, as a result of their thorough discussion, to reflect in the final specifications the best current practice.

The Joint Committee further calls attention to the fact that it has undertaken to prepare specifications covering the fundamentals to be observed in the general use of concrete and reinforced concrete; no attempt has been made to cover the details involved in the use of these materials in special structures. While the sections relating to design deal primarily with building construction, nevertheless the principles involved are in general applicable to structures of other types. It is expected that in using these specifications the necessary supplemental requirements will be added covering details.

This report has been submitted to letter ballot of the Joint Committee which consists of 25 members, representing 5 societies, all of whom have voted affirmatively.

Respectfully submitted,

RICHARD L. HUMPHREY, *Chairman.*

J. J. YATES, *Vice-Chairman.*
DUFF A. ABRAMS, *Secretary-Treasurer.*

ERNEST ASHTON,
GEORGE E. BOYD,
EDWARD D. BOYER,
J. E. FREEMAN,
ALBERT T. GOLDBECK,
WILLIAM K. HATT,
S. C. HOLLISTER,
EDWARD E. HUGHES,
FREDERICK W. KELLEY,
ROBERT W. LESLEY,
A. E. LINDAU,
ARTHUR R. LORD,
ANGUS B. MACMILLAN,
FRANKLIN R. MCMILLAN,
LEON S. MOISSEIFF,
EGBERT J. MOORE,
HENRY H. QUIMBY,
FREDERICK E. SCHALL,
W. A. SLATER,
SANFORD E. THOMPSON,
H. T. WELTY,
C. C. WESTFALL.

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LIST OF TABLES.

LIST OF APPENDICES.

The specifications and methods of test given in Appendices III to XV, inclusive, form a part of the Tentative Specifications for Concrete and Reinforced Concrete,* but are not reproduced here.

* These specifications and methods of test are those of the American Society for Testing Materials, either in their present form as adopted by the Society or in the form in which the respective committees of the Society will recommend them for action at the Annual Meeting of the Society, June 21st-24th, 1921.

† American Society for Testing Materials, 1921 Book of A. S. T. M. Standards.

‡ American Society for Testing Materials, *Proceedings*, Vol. XXI, Part I (1921).

TENTATIVE SPECIFICATIONS

FOR

CONCRETE AND REINFORCED CONCRETE.

CHAPTER I.

General Instructions.

1.—These specifications are not complete; they cover the general conditions affecting the use of concrete and reinforced concrete. To complete them it will be necessary for the Engineer to

(*a*) Provide the detail specifications covering the work in particular in which the concrete and reinforced concrete are to be used;

(*b*) Insert in Section 4 the strengths required for the several classes of concrete specified, based either upon preliminary tests or upon the values given in Table 4;

(*c*) Insert in Section 14 the sizes of aggregates required;

(*d*) Strike out one of the titles of the specifications in Section 20;

(*e*) Strike out one of the titles of the specifications in Section 24;

(*f*) Strike out one of the words "volume" or "weight" in Section 27;

(*g*) Strike out two of the three Sections 28 and fill in the necessary blanks for the proportions;

(*h*) Insert in Section 29 the slumps required;

(*i*) Strike out the method or methods inapplicable to the work, in Section 50;

(*j*) Strike out one of the two Sections 97.

CHAPTER II.

Definitions.

2.—The following definitions give the meaning of certain terms as used in these specifications:

Acid Proofing.—Treatment of a concrete surface to resist the action of acid solutions.

Aggregate.—Inert material which is mixed with Portland cement and water to produce concrete; in general, aggregate consists of sand, pebbles, gravel, crushed stone or gravel, or similar materials. (See *Fine Aggregate; Coarse Aggregate.*)

Approved.—Meeting the approval of, or specifically authorized by, the Engineer.

Buttressed Retaining Wall.—A reinforced concrete wall having a vertical stem and a horizontal base, with brackets on the side opposite the pressure face uniting the vertical section with the toe of the base.

Cantilever Retaining Wall.—A reinforced concrete wall having a vertical stem and a horizontal base, each of which resists by cantilever action the pressure to which it is subjected.

Cellular Retaining Wall.—A reinforced concrete wall with a horizontal base, longitudinal vertical sections, and a series of transverse walls, dividing

the space between the longitudinal walls into cells which are filled with earth, or other suitable material. If the top of the cells is covered by a floor-slab, the front longitudinal wall and the filling may be omitted.

Coarse Aggregate.—Aggregate retained on a No. 4 sieve and of a maximum size generally not larger than 3 in. (See *Aggregate; Fine Aggregate*).

Column.—A vertical compression member whose length exceeds three times its least horizontal dimension.

Column Capital.—An enlargement of the upper end of a reinforced concrete column built monolithic with the column and flat slab to increase the moment of inertia of the column and the shearing resistance of the slab at sections where high bending moment or high shear may occur.

Column Strip.—A portion of a panel of a flat slab which has a uniform width equal to one-fourth of the panel length on a line perpendicular to the direction of the strip, and whose outer edge lies on the edge of the panel. (See *Middle Strip*).

Concrete.—A mixture of Portland cement, fine aggregate, coarse aggregate, and water. (See *Mortar*).

Consistency.—A general term used to designate the relative plasticity of freshly mixed mortar and concrete.

Counterforted Retaining Wall.—A reinforced concrete wall having a vertical stem and a horizontal base with brackets on the pressure face uniting the vertical section with the heel of the base.

Crusher-Run Stone.—Unscreened crushed stone. (See *Stone Screenings*).

Cyclopean Concrete.—Concrete in which stones larger than one-man size are individually embedded.

Dead Load.—The weight of the structure plus fixed loads and forces.

Deformed Bar.—Reinforcement bar with shoulders, lugs, or projections formed integrally from the body of the bar during rolling.

Diagonal Direction.—A direction parallel or approximately parallel to the diagonal of the panel.

Dropped Panel.—The structural portion of a flat slab which is thickened throughout an area surrounding the column capital.

Effective Area of Concrete.—The area of a section of the concrete which lies between the tension reinforcement and the compression surface of the beam or slab.

Effective Area of Reinforcement.—The area obtained by multiplying the right cross-sectional area of the metal reinforcement by the cosine of the angle between the direction of the reinforcement bars or wires, and the direction for which the effectiveness of the reinforcement is to be determined.

Engineer.—The engineer in responsible charge of design and construction.

Fine Aggregate.—Aggregate passing through a No. 4 sieve. (See *Aggregate; Coarse Aggregate*).

Flat Slab.—A flat concrete floor or roof plate having reinforcement bars extending in two or more directions and having no beams or girders to carry the load to the supporting columns.

Footing.—A structural unit used to distribute wall or column loads to the supporting material, either directly or through piles.

Gravel.—Loose material containing particles larger than sand, resulting from natural crushing and erosion of rocks. (See *Sand*).

Laitance.—The extremely fine particles which separate from freshly deposited mortar or concrete and collect on the top surface.

Live Load.—Loads and forces which are variable.

Membrane Water-Proofing.—A coating reinforced by fabric, felt, or similar toughening material applied to structures to prevent contact of moisture.

Middle Strip.—The portion of a panel of a flat slab which extends in a direction parallel to a side of the panel, whose width is one-half the panel length on a line at right angles to the direction of the strip and whose center line lies on the center line of the panel. (See *Column Strip*).

Mortar.—A mixture of Portland cement, fine aggregate, and water. (See *Concrete*).

Negative Reinforcement.—Reinforcement so placed as to take stress due to negative bending moment.

Oil-Proofing.—Treatment of a concrete surface to resist the action of mineral, animal, or vegetable oils.

One-Man Stone.—Stone larger than coarse aggregate and not exceeding 100 lb. in weight. (See *Rubble Concrete*).

Panel Length.—The distance between centers of two columns of a panel, in either rectangular direction.

Pedestal Footing.—A member supporting a column, in which the projection from the face of the column on all sides is less than one-half the depth.

Pedestal or Pier.—A vertical compression member whose length does not exceed three times its least horizontal dimension.

Plain Concrete.—Concrete without metal reinforcement.

Portland Cement.—The product obtained by finely pulverizing clinker produced by calcining to incipient fusion an intimate and properly proportioned mixture of argillaceous and calcareous materials, with no additions subsequent to calcination excepting water and calcined or uncalcined gypsum.

Positive Reinforcement.—Reinforcement so placed as to take stress due to positive bending moment.

Principal Design Section.—The vertical sections in a flat slab on which the moments in the rectangular directions are critical. (See Section 146).

Ratio of Reinforcement.—The ratio of the effective area of the reinforcement cut by a section of a beam or slab to the effective area of the concrete cut by that section.

Rectangular Direction.—A direction parallel to a side of the panel.

Reinforced Concrete.—Concrete in which metal is embedded in such a manner that the two materials act together in resisting stress.

Rubble Aggregate.—Stone or gravel larger than coarse aggregate and not larger than one-man stone. (See *One-Man Stone*).

Rubble Concrete.—Concrete in which pieces of rubble aggregate are individually embedded. (See *Rubble Aggregate*).

Sand.—Loose material consisting of small grains (commonly quartz) resulting from the natural disintegration of rocks. (See *Gravel*).

Screen.—A metal plate with closely spaced circular perforations. (See *Sieve*).

Sieve.—Woven wire cloth with square openings. (See *Screen*).

Slump.—The shortening of a standard test mass of concrete used as a measure of workability.

Standard Sand.—Natural sand mined at Ottawa, Ill., screened to pass a No. 20 sieve and retained on a No. 30 sieve, used as the fine aggregate in standard strength tests of Portland cement. (See Appendix III for Specifications).*

Stone Screenings.—Unscreened crushed stone passing through a No. 4 sieve. (See *Crusher-Run Stone*).

Tremie.—A water-tight pipe of suitable dimensions, generally used in a vertical position, for depositing concrete under water.

Wall-Beam.—A reinforced concrete beam which extends from column to column along the outer edge of a wall panel.

CHAPTER III.

QUALITY OF CONCRETE.

3.—*Quality.*—The quality of concrete shall be expressed in terms of workability as determined by the slump test, and of the compressive strength at 28 days as determined by concrete tests of the materials to be used, as specified in Section 28. The proportions required to produce concrete having the strength specified in Section 4 shall be determined in advance of the mixing of the concrete.

4.—*Strength.*—The concrete shall develop under the conditions specified in Section 3, for the various parts of the work, the following strengths:†

.... lb. per sq. in.
.... lb. per sq. in.
.... lb. per sq. in.
.... lb. per sq. in.

5.—*Tests of Field Specimens.*—Field concrete test specimens shall be made, stored, and tested in accordance with "Standard Methods of Making and Storing Specimens of Concrete in the Field" (Serial Designation: C31—21) of the American Society for Testing Materials. (Appendix XIV).*

CHAPTER IV.

MATERIALS.

A.—PORTLAND CEMENT.

6.—Portland cement shall conform to the "Standard Specifications and Tests for Portland Cement"‡ (Serial Designation: C9—21) of the American Society for Testing Materials (Appendix III)*, and subsequent revisions thereof.

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* Not reproduced.

† The engineer should insert the strengths for the several classes of concrete specified, based either upon preliminary tests or upon the values given in Table 4, p. 111.

‡ These specifications are also a standard of the following organizations: American Engineering Standards Committee, United States Government, American Railway Engineering Association, American Concrete Institute, and the Portland Cement Association.

B.—Fine Aggregate.

7.—*General Requirements.*—Fine aggregate shall consist of sand, stone screenings, or other inert materials with similar characteristics, or a combination thereof, having clean, hard, strong, durable, uncoated grains and free from injurious amounts of dust, lumps, soft or flaky particles, shale, alkali, organic matter, loam, or other deleterious substances.

8.—*Grading.*—Fine aggregate shall range in size from fine to coarse, preferably within the following limits:

Passing through No. 4 sieve....not less than 95 per cent.
Passing through No. 50 sieve....not more than 30 per cent.
Weight removed by decantation....not more than 3 per cent.

9.—*Sieve Analysis.*—The sieves and method of making sieve analysis shall conform to the "Tentative Method of Test for Sieve Analysis of Aggregates for Concrete" (Serial Designation: C41—21T) of the American Society for Testing Materials. (Appendix IX)*.

10.—*Decantation Test.*—The decantation test shall be made in accordance with the "Standard Method of Test for Quantity of Clay and Silt in Sand for Highway Construction" (Serial Designation: D74—21) of the American Society for Testing Materials. (Appendix X.)*

11.—*Mortar Strength Test.*—Fine aggregate shall preferably be of such a quality that mortar briquettes, cylinders, or prisms, consisting of one part by weight of Portland cement and three parts by weight of fine aggregate†, mixed and tested in accordance with the methods described in the "Standard Specifications and Tests for Portland Cement" (Appendix III),* will show a tensile or compressive strength at ages of 7 and 28 days not less than that of 1:3 standard Ottawa sand mortar of the same plasticity made with the same cement. However, fine aggregate which fails to meet this requirement may be used, provided the proportions of cement, fine aggregate, coarse aggregate, and water are such as to produce concrete of the strength specified.‡ Concrete tests shall be made in accordance with the "Tentative Methods for Making Compression Tests of Concrete" (Serial Designation: C39—21T) of the American Society for Testing Materials. (Appendix XIII).*

12.—*Organic Impurities in Sand.*—Natural sand which shows a color darker than the standard color when tested in accordance with the "Tentative Method of Test for Organic Impurities in Sand for Concrete" (Serial Designation: C40—21T) of the American Society for Testing Materials (Appendix XI),* shall not be used, unless the concrete made with the materials and in the proportions to be used on the work is shown by tests to be of the required strength.

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* Not reproduced.

† In testing aggregate, care should be exercised to avoid the removal of any coating on the grains which may affect the strength. Natural sand should not be dried before being made into mortar, but should contain natural moisture. The quantity of water contained may be determined on a separate sample and the weight of the sand used in the test corrected for the moisture content.

‡ Table 4 furnishes a guide in determining the proportions of materials required to produce a concrete of a given strength, using aggregates of various sizes and concrete of different consistencies.

C.—Coarse Aggregate.

13.—*General Requirements*.—Coarse aggregate shall consist of crushed stone, gravel, or other approved inert materials with similar characteristics, or combinations thereof, having clean, hard, strong, durable, uncoated particles free from injurious amounts of soft, friable, thin, elongated or laminated pieces, alkali, organic, or other deleterious matter.

14.—*Grading*.—Coarse aggregate shall range in size from fine to coarse within the following limits*:

Passing	† in.	sieve (maximum size)....	not more than 95	per cent.
Passing	† in.	" (intermediate size)...	† to †	per cent.
Passing	No. 4	"	not more than 15	per cent.
Passing	No. 8	"	" " " 5	per cent.

15.—*Sieve Sizes*.—The test for size and grading of aggregate shall be made in accordance with the "Tentative Method of Test for Sieve Analysis of Aggregate for Concrete" (Serial Designation: C41—21T) of the American Society for Testing Materials. (Appendix IX).‡

D.—Rubble and Cyclopean Aggregate.

16.—*Rubble Aggregate*.—Rubble aggregate shall consist of clean, hard, durable stone larger than coarse aggregate and not larger than one-man stone.

17.—*Cyclopean Aggregate*.—Cyclopean aggregate shall consist of clean, hard, durable stone, free from fissures and planes of cleavage and larger than one-man stone.

E.—Storage of Aggregate.

18.—*Aggregate Storage*.—Aggregate shall be so stored on platforms or otherwise as to avoid the inclusion of foreign materials. Before using, frost, ice, and lumps of frozen materials shall be removed.

F.—Water.

19.—*General Requirements*.—Water for concrete shall be clean and free from oil, acid, alkali, organic matter, or other deleterious substance.

* Where several suitable aggregates are available, a thorough investigation of the relative economy of each for producing concrete of the desired strength is advisable, especially for work of considerable magnitude.

† The engineer should insert in these blanks the sizes of aggregates required. The size and grading to be used will be governed by local conditions. The limitation on size and grading is intended to secure uniformity of aggregate. The following table indicates desirable gradings for coarse aggregate for certain maximum sizes:

Maximum size of aggregate, in inches.	Percentage by Weight Passing through Standard Sieves with Square Openings.					Percentage Passing, not more than	
	3 in.	2 in.	1½ in.	1 in.	¾ in.	No. 4 sieve.	No. 8 sieve.
3	100	....	40-75	....	....	15	5
2	....	100	....	40-75	....	15	5
1½	....	....	100	....	40-75	15	5
1	....	....	....	100	....	15	5
¾	...	....	....	....	100	15	5

‡ Not reproduced.

G.—Metal Reinforcement.

20.—*Quality.*—Metal reinforcement shall be of a quality and character meeting the requirements of the "Standard Specifications* for Billet-Steel Concrete Reinforcement Bars" (Serial Designation: A15—14), of the American Society for Testing Materials (Appendix IV)†, "Standard Specifications* for Rail-Steel Concrete Reinforcement Bars" (Serial Designation: A16—14), of the American Society for Testing Materials (Appendix V)†, except that the provision for machining deformed bars before testing shall be eliminated.

21.—*Wire.*—Wire for concrete reinforcement shall conform to the requirements of the "Tentative Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement" (Serial Designation: A82—21T), of the American Society for Testing Materials. (Appendix VIII).†

22.—*Standard Sizes of Bars.*—Reinforcement bars shall conform to the areas and equivalent sizes shown in Table 1.

The areas of deformed bars shall be determined by the minimum cross-section thereof.

TABLE 1.—Sizes and Areas of Reinforcement Bars.

Size of bar, in inches.	Area, in Square Inches.	
	Round.	Square.
⅜	0.110	.....
½	0.196	0.250
⅝	0.307	.....
¾	0.442	.....
⅞	0.601	.....
1	0.785	1.000
1⅛	.....	1.266
1¼	.....	1.563

23.—*Deformed Bars.*—An approved deformed bar shall be one that will develop a bond strength at least 25% greater than that of a plain round bar of equivalent cross-sectional area.‡

24.—*Structural Shapes.*—Structural steel shapes used for reinforcement shall conform to the requirements of "Standard Specifications§ for Structural Steel for Bridges" (Serial Designation: A7—21), of the American Society for Testing Materials (Appendix VI)†, "Standard Specifications§ for Structural Steel for Buildings" (Serial Designation: A9—21), of the American Society for Testing Materials (Appendix VII).†

25.—*Cast Iron.*—The quality of cast iron used in composite columns shall conform to the requirements of the "Standard Specifications for Cast-Iron Pipe and Special Castings" (Serial Designation: A44—04), of the American Society for Testing Materials. (Appendix XV).†

* The engineer should strike out one of these titles. The Committee recommends as preferred material for reinforcement that meeting the requirements of the "Standard Specifications for Billet-Steel Concrete Reinforcement Bars" (Serial Designation: A15–14), of intermediate grade (except as noted under Section 20), made by the open-hearth process.

† Not reproduced.

‡ The Committee has under consideration a specification for deformed bars, but is not prepared at this time to make more definite recommendations.

§ The engineer should strike out one of these titles.

CHAPTER V.

Proportioning and Mixing Concrete.

A.—Proportioning.

26.—*Unit of Measure.*—The unit of measure shall be the cubic foot. Ninety-four (94) pounds (one bag or ¼ bbl.) of Portland cement shall be considered as one cubic foot.

27.—*Method of Measuring.*—Each of the constituent materials shall be measured separately by volume* weight.* The method of measurement shall be such as to secure the specified proportions in each batch. If volume measurement is used, the fine aggregate and the coarse aggregate shall be measured loose as thrown into the measuring device. The water shall be measured by an automatic device that will insure the same quantity in successive batches.

28†.—*Proportions.*—The proportions of cement, water, and aggregate shall be such as to produce concrete of the strength and quality specified in Sections 3 and 4. The proportions shall be 1 part of Portland cement....‡ parts of fine aggregate, and....‡ parts of coarse aggregate as determined by the Engineer from concrete tests of the materials to be used. The tests shall be made in accordance with the "Tentative Methods of Making Compression Tests of Concrete" (Serial Designation: C39—21T) of the American Society for Testing Materials. (Appendix XIII)‖. The quantity of water used shall be such as to produce concrete of the consistency required by the particular class of work, and shall be as specified in Section 29. In case the grading of the supply of available aggregate varies from that upon which the proportions were based, such aggregate may be used, provided the new proportions, as determined by the Engineer, are such as to produce concrete of the required strength and quality.

28†.—*Proportions.*—The contractor shall use materials so proportioned and mixed as to produce concrete of the required workability and strength§. Frequent compression tests of the concrete used in the work will be made by the Engineer, and in case of failure to meet the specified strength, the contractor shall make such changes in the materials, proportions, or mixing, as may be necessary to secure concrete of the required strength. Concrete tests shall be made in accordance with the "Standard Methods of Making and Storing Specimens of Concrete in the Field" (Serial Designation: C31—21), of the American Society for Testing Materials (Appendix XIV),‖ and the "Tentative Methods of Making Compression Tests of Concrete" (Appendix XIII)‖.

28†.—*Proportions.*—The proportions shall be 1 part of Portland cement...‡ parts of fine aggregate and....‡ parts of coarse aggregate. The proportions of

* The engineer should strike out one of these terms.

† The engineer should indicate his choice of the method of proportioning to be used by striking out two of the Sections numbered 28.

‡ The engineer should fill in these blanks.

§ The use of this method should be accompanied by a clause in the contract which indicates the procedure to be followed in case tests show that concrete of the specified strength has not been obtained.

‖ Not reproduced.

materials shall be selected from Table 4. In case the grading of the supply of available aggregate varies from that upon which the proportions were based, such aggregate may be used, provided the new proportions, as determined by the Engineer, are such as to produce concrete of the required strength and quality.

B.—Consistency.

29.—The Engineer shall determine and specify the consistency of the concrete for various portions of the work based on tests of the materials to be used. The consistency of the concrete shall be measured by the slump test in the manner described in the "Tentative Specifications for Workability of Concrete for Concrete Pavements" (Serial Designation: D62—20T), of the American Society for Testing Materials. (Appendix XII)*. The slump for different types of concrete shall not be greater than as indicated in Table 2.

The consistency shall be checked from time to time during the progress of the work.

TABLE 2.—Workability of Concrete.

Type of Concrete.	Maximum slump, in inches.
1.—Mass concrete	†
2.—Reinforced concrete:	
(*a*) Thin vertical sections and columns	†
(*b*) Heavy sections	†
(*c*) Thin confined horizontal sections	†
3.—Roads and pavements:	
(*a*) Hand-finished	†
(*b*) Machine finished	†
4.—Mortar for floor finish	†

† The engineer should insert the slumps required, based on tests called for in this Section. The slump test requirement is intended to insure concrete mixed with the minimum quantity of water required to produce a plastic mixture. The following table indicates the maximum slump desirable for the various types of concrete, based on average aggregates and proportions:

Type of Concrete.	Maximum slump, in inches.
1.—Mass concrete	2
2.—Reinforced concrete:	
(*a*) Thin vertical sections and columns	6
(*b*) Heavy sections	2
(*c*) Thin confined horizontal sections	8
3.—Roads and pavements:	
(*a*) Hand-finished	4
(*b*) Machine-finished	1
4.—Mortar for floor finish	2

C.—Mixing.

30.—*Machine-Mixing.*—Mixing, unless otherwise authorized by the Engineer, shall be done in a batch mixer of approved type, which will insure a uniform distribution of the materials throughout the mass, so that the mixture is uniform in color and homogeneous. The mixer shall be equipped with suitable charging hopper, water storage, and a water-measuring device controlled from a case which can be kept locked and so constructed that the water can be discharged only while the mixer is being charged. It shall also be

* Not reproduced.

equipped with an attachment for automatically locking the discharge lever until the batch has been mixed the required time after all materials are in the mixer. The entire contents of the drum shall be discharged before recharging. The mixer shall be cleaned at frequent intervals while in use.

31.—*Time of Mixing.*—The mixing of each batch shall continue not less than 1½ min. after all the materials are in the mixer, during which time the mixer shall rotate at a peripheral speed of about 200 ft. per min. The volume of the mixed material per batch shall not exceed the manufacturer's rated capacity of the mixer.

32.—*Hand-Mixing.*—When hand-mixing is authorized by the Engineer it shall be done on a water-tight platform. The materials shall be turned at least six times after the water is added and until the batch is homogeneous in appearance and color.

33.—*Retempering.*—The retempering of concrete or mortar which has partly hardened, that is, remixing with or without additional cement, aggregate, or water, shall not be permitted.

CHAPTER VI.

Depositing Concrete.

A.—Depositing in Air.

34.—*General.*—Before beginning a run of concrete, hardened concrete and foreign materials shall be removed from the inner surfaces of mixing and conveying equipment.

35.—*Approval.*—Before depositing concrete, débris shall be removed from the space to be occupied by the concrete; forms shall be thoroughly wetted (except in freezing weather), or oiled. Reinforcement shall be thoroughly secured in position and approved by the Engineer.

36.—*Handling.*—Concrete shall be handled from the mixer to the place of final deposit as rapidly as practicable by methods which shall prevent the separation or loss of the ingredients. It shall be deposited in the forms as nearly as practicable in its final position to avoid rehandling. It shall be deposited in approximately uniform horizontal layers; the piling up of the concrete in the forms in such manner as to permit the escape of the mortar from the coarse aggregate will not be permitted. Forms for walls or other thin section of considerable height, shall be provided with openings, or other devices which will permit the concrete to be placed in a manner that will avoid accumulations of hardened concrete on forms or metal reinforcement. Under no circumstances shall concrete that has partly hardened be deposited in the work.

37.—*Spouting.*—Where concrete is conveyed by spouting, the plant shall be of such size and design as to ensure a practically continuous flow in the spout. The angle of the spout with the horizontal shall be such as to allow the concrete to flow without separation of the ingredients.* The spout

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*** An angle of about 27°, or one vertical to two horizontal, is good practice. Spouting through a vertical pipe is satisfactory when the flow is continuous; when it is unchecked and discontinuous it is highly objectionable unless the flow is broken by baffles.**

shall be thoroughly flushed with water before and after each run. The delivery from the spout shall be as close as possible to the point of deposit. When operation must be intermittent, the spout shall discharge into a hopper.

38.—*Compacting.*—Concrete, during and immediately after depositing, shall be thoroughly compacted by means of rods or forks. For thin walls or inaccessible portions of the forms where rodding or forking is impracticable, the concrete shall be assisted into place by tapping or hammering the forms. The concrete shall be thoroughly worked around the reinforcement, and around embedded fixtures, into the corners of the forms.

39.—*Removal of Water.*—Water shall be removed from excavations before concrete is deposited, unless otherwise directed by the Engineer. A continuous flow of water into the excavation shall be diverted through proper side-drains to a sump, or by other approved methods which will avoid washing the freshly deposited concrete.

40.—*Protection.*—Exposed surfaces of concrete subjected to premature drying shall be kept thoroughly wetted for a period of at least 7 days.

41.—*Cold Weather.*—Concrete mixed and deposited during freezing weather shall have a temperature of not less than 50° Fahr., nor more than 100° Fahr. Suitable means shall be provided for maintaining a temperature of at least 50° Fahr. for not less than 72 hours after placing, or until the concrete has thoroughly hardened. The methods of heating the materials and protecting the concrete shall be approved by the Engineer. Salt, chemicals, or other foreign materials shall not be used to prevent freezing.

42.—*Depositing Continuously.*—Concrete shall be deposited continuously and as rapidly as practicable and until the unit of operation, as approved by the Engineer, is completed. Construction joints at points not provided for in the plans, shall be made in accordance with the provisions in Section 69.

43.—*Bonding.*—The surface of the hardened concrete shall be roughened and thoroughly cleaned of foreign matter and laitance, and saturated with water and forms retightened before depositing concrete. An excess of mortar on vertical or inclined surfaces shall be secured by thoroughly rodding or forking the freshly deposited concrete to remove the coarse aggregate from contact with the hardened concrete.

B.—Rubble and Cyclopean Concrete.

44.—*Rubble Concrete.*—Rubble aggregate shall be thoroughly embedded in the concrete. The individual stones shall not be closer to any surface or adjacent stone than the maximum size of the coarse aggregate plus 1 in. Each successive layer of concrete shall be keyed in accordance with the provision in Section 69.

45.—*Cyclopean Concrete.*—Cyclopean aggregate shall be thoroughly embedded in the concrete; no stone shall be closer to a finished surface than 1 ft., nor closer than 6 in. to any adjacent stone. Stratified stone shall be laid on its natural bed.

C.—Depositing Under Water.*

46.—*General.*—The methods, equipment, and materials to be used shall be submitted to and approved by the Engineer before the work is started. Concrete shall be deposited by a method that will prevent the washing of the cement from the mixture, minimize the formation of laitance, and avoid flow of water until the concrete has fully hardened. Concrete shall be placed so as to minimize segregation of materials. Hand-mixing will not be permitted. Concrete shall not be placed in water at temperatures below 35° Fahr.

47.—*Proportions.*—Concrete deposited under water shall consist of not less than 1 part of Portland cement to 6 parts of fine and coarse aggregate, measured separately.

48.—*Coffer-dams.*—Coffer-dams shall be sufficiently tight to prevent flow of water through the space in which concrete is to be deposited. Pumping will not be permitted while concrete is being deposited, nor until it has fully hardened.

49.—*Depositing Continuously.*—Concrete shall be deposited continuously, keeping the top surface as nearly level as possible, until it is brought above water, or to the required height. The work shall be carried on with sufficient rapidity to insure bonding of the successive layers.

50.—*Method.*—The following method† shall be used for depositing concrete under water:

(*a*) *Tremie.*—The tremie shall be water-tight and sufficiently large to permit a free flow of concrete. It shall be kept filled‡ at all times during depositing. The concrete shall be discharged and spread by raising the tremie in such manner as to maintain as nearly as practicable a uniform flow and avoid dropping the concrete through water. If the charge is lost during depositing the tremie shall be withdrawn and refilled.

(*b*) *Drop-Bottom Bucket.*—The bucket shall be of a type that cannot be dumped until it rests on the surface upon which the concrete is to be deposited. The bottom doors when tripped shall open freely downward and outward. The top of the bucket shall be open. The bucket shall be completely filled, and slowly lowered to avoid back-wash. When discharged, the bucket shall be withdrawn slowly until clear of the concrete.

(*c*) *Bags.*—Bags of jute or other coarse cloth shall be filled about two-thirds full of concrete and carefully placed by hand in a header-and-stretcher system so that the whole mass is interlocked.

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* Concrete should not be deposited under water if practicable to deposit in air. There is always uncertainty as to the results obtained from placing concrete under water; where conditions permit, the additional expense and delay of avoiding this method will be warranted. It is especially important that the aggregate be free from loam and other material which may cause laitance. Washed aggregates are preferable. Coarse aggregate consisting of washed gravel of a somewhat smaller size used in open-air concrete work will give the best results. Concrete should never be deposited under water without experienced supervision. Many failures, especially of structures in sea water, can be traced directly to ignorance of proper methods or lack of expert supervision.

† The engineer should strike out the method or methods inapplicable to the work.

‡ The tremie may be filled by one of the following methods: (1) Place the lower end in a box partly filled with concrete, so as to seal the bottom, then lower into position; (2) plug the tremie with cloth sacks or other material, which will be forced down as the tube is filled with concrete; (3) plug the end of the tremie with cloth sacks filled with concrete.

51.—*Laitance.*—The concrete shall be disturbed as little as possible while it is being deposited, in order to avoid the formation of laitance. Laitance shall be removed.

CHAPTER VII.

Forms.

52.—*General.*—Forms shall conform to the shape, lines, and dimensions of the concrete as called for on the plans. Lumber used in forms for exposed surfaces shall be dressed to a uniform thickness, and shall be free from loose knots or other defects. Joints in forms shall be horizontal or vertical. For unexposed surfaces and rough work, undressed lumber may be used. Lumber once used in forms shall have nails withdrawn, and surfaces to be in contact with concrete thoroughly cleaned, before being used again.

53.—*Design.*—Forms shall be substantial and sufficiently tight to prevent leakage of mortar; they shall be properly braced or tied together so as to maintain position and shape. If adequate foundation for shores cannot be secured, trussed supports shall be provided.

54.—*Workmanship.*—Bolts and rods shall preferably be used for internal ties; they shall be so arranged that when the forms are removed no metal shall be within 1 in. of any surface. Wire ties will be permitted only on light and unimportant work; they shall not be used through surfaces where discoloration would be objectionable. Shores supporting successive stories shall be placed directly over those below, or so designed that the load will be transmitted directly to them. Forms shall be set to line and grade and so constructed and fastened as to produce true lines. Special care shall be used to prevent bulging.

55.—*Mouldings.*—Unless otherwise specified, suitable mouldings or bevels shall be placed in the angles of forms to round or bevel the edges of the concrete.

56.—*Oiling.*—The inside of forms shall be coated with non-staining mineral oil, or other approved material, or thoroughly wetted (except in freezing weather). Where oil is used, it shall be applied before the reinforcement is placed.

57.—*Inspection of Forms.*—Temporary openings shall be provided at the base of column and wall forms, and other places where necessary, to facilitate inspection and cleaning immediately before depositing concrete.

58.—*Removal of Forms.*—Forms shall not be disturbed until the concrete has adequately hardened, nor shall the permanent shores be removed until the structure has attained its full design strength* and all excess construction load has been removed. Wall and column forms shall be left in place until the concrete has hardened sufficiently to sustain its own weight and the construction loads likely to come upon it. Forms other than wall or column forms shall be left in place until the concrete has hardened sufficiently to carry the full load which it must sustain, unless removed in sections and each section of the structure is immediately re-shored.

* Many conditions affect the hardening of concrete, and the proper time for the removal of the forms should be determined by a competent and responsible person.

CHAPTER VIII.

Details of Construction.

A.—Metal Reinforcement.

59.—*Cleaning.*—Metal reinforcement, before being positioned, shall be thoroughly cleaned of mill and rust scale, and of coatings of any character that will destroy or reduce the bond. Reinforcement appreciably reduced in section shall be rejected. Reinforcement shall be re-inspected and when necessary cleaned where there is delay in depositing concrete.

60.—*Bending.*—Reinforcement shall be carefully formed to the dimensions indicated on the plans or called for in the specifications. The radius of bends shall be four or more times the least diameter of the reinforcement bar.

61.—*Straightening.*—Metal reinforcement shall not be bent or straightened in a manner that will injure the material. Bars with kinks or bends shall not be used.

62.—*Placing.*—Metal reinforcement shall be accurately positioned, and secured against displacement by using annealed iron wire of not less than No. 18 gauge, or suitable clips at intersections, and shall be supported by concrete or metal chairs, or spacers, or by metal hangers. Parallel bars shall not be placed closer in the clear than 1½ times the diameter of round bars or 1½ times the diagonal of square bars; if the ends of bars are hooked as specified in Section 130, the clear spacing may be made equal to the diameter of the round bars or to the diagonal of square bars, but in no case shall the spacing between bars be less than 1 in., nor less than 1¼ times the maximum size of the coarse aggregate.

63.—*Splicing.*—Splices of tensile reinforcement at points of maximum stress shall be avoided. Splices, where required, shall provide sufficient lap to transfer the stress between bars by bond and shear, or by a mechanical connection such as a screw coupling.

64.—*Offsets in Column Reinforcement.*—Vertical reinforcement shall be offset in a region where lateral support is afforded when changes in column cross-section occur and the vertical reinforcement bars are not sloped for the full length of the column.

65.—*Future Bonding.*—Exposed reinforcement bars intended for bonding with future extensions shall be protected from corrosion.

B.—Concrete Covering Over Metal.

66.—*Moisture Protection.*—Metal reinforcement in wall footings and column footings shall have a minimum covering of 3 in. of concrete.

67.—*Fire Protection.*—Metal reinforcement in fire-resistive construction shall be protected by not less than 1 in. of concrete in slabs and walls, and not less than 2 in. in beams, girders, and columns, provided aggregate showing an expansion not materially greater than that of limestone or trap rock is used; when impracticable to obtain aggregate of this grade, the protective covering shall be 1 in. thicker and shall be reinforced with metal mesh not exceeding 3 in. in greatest dimensions, placed 1 in. from the finished surface.

The metal reinforcement in structures containing incombustible materials and in bridges where the fire hazard is limited, shall be protected by not less than $\frac{3}{4}$ in. of concrete in slabs and walls and of not less than $1\frac{1}{2}$ in. in beams, girders, and columns.

68.—*Plaster.*—Plaster finish on an exposed concrete surface may be allowed to reduce the thickness of concrete protection called for in Section 67, by half the thickness of the plaster, but the protection shall not be less than that specified in Sections 66 and 67.

C.—JOINTS.

69.—*Construction Joints.*—Construction joints not indicated on the plans nor specified shall be located and formed so as to least impair the strength and appearance of the structure. Horizontal construction joints shall be formed by embedding stones projecting above the surface, or by roughening the surface in contact, or by mortises or keys formed in the concrete. Sufficient section shall be provided in horizontal as well as vertical keys to resist shear.

70.—*Joints in Columns.*—Construction joints in columns shall be made at the under side of the floor. Haunches and column capitals shall be considered as part of, and built monolithic with, the floor construction.

71.—*Joints in Floors.*—Construction joints in floors shall be located near the center of spans of slabs, beams, and girders, unless a beam intersects a girder at this point, in which case the joints in the girders shall be offset a distance equal to twice the width of the beam. Adequate provision shall be made for shear either by sufficient reinforcement, or by sloping the joint so as to provide an inclined bearing.

72.—*Monolithic Construction.*—Girders and beams designed to be monolithic with walls and columns shall not be cast until 2 hours after the completion of the walls or columns.

73.—*Construction Joints in Long Buildings.*—Construction joints made cross-wise of a building 100 ft. or more in length, shall have special reinforcement placed at right angles to the joint and extending a sufficient distance on each side of the joint to develop the strength of the reinforcement by bond. This reinforcement shall be placed near the opposite face of the member from the main tension reinforcement; the amount of such reinforcement shall be not less than 0.5% of the section of the members cut by the joint.

74.—*Expansion Joints.*—Expansion joints shall be so detailed that the necessary movement may occur with the minimum of resistance at the joint. The structure adjacent to the joint shall preferably be supported on separate columns or walls. Reinforcement shall not extend across an expansion joint. The break between the two sections shall be complete, and may be effected by a coating of white lead and oil, asphalt paint or petrolatum, or by building paper, placed over the entire surface of the hardened concrete. Exposed edges of expansion joints in walls or abutments shall be bonded. Exposed expansion joints formed between two distinct concrete members shall be filled with an elastic joint filler of approved quality.

75.—*Expansion Joints in Long Buildings.*—Structures exceeding 200 ft. in length and of width less than about one-half the length, shall be divided by means of expansion joints, located near the middle, but not more than 200 ft. apart, to minimize the destructive effects of temperature changes and shrinkage. Structures in which marked changes in plan section take place abruptly, or within a small distance, shall be provided with expansion joints at the points where such changes in section occur.

76.—*Sliding Joints.*—The seat of sliding joints shall be finished with a smooth troweled surface and shall not have the superimposed concrete placed upon it until it has thoroughly hardened. In order to facilitate sliding, two thicknesses of building paper shall be placed over the seat on which the superimposed concrete is to be deposited.

77.—*Water-Tight Joints.*—When it is not possible to finish a section of the structure in one continuous operation and water-tight construction is required, the joints shall be prepared as follows: the surface of the first section of concrete shall be provided with continuous keyways. All laitance and other foreign substances shall be removed from the surface of the concrete first placed; this surface shall then be thoroughly saturated with water and given a heavy coating of neat cement. The next section of concrete shall be placed in such manner as to insure an excess of mortar over the entire surface of the joint. Where shown on the plans, the joint shall be so constructed as to permit of its being caulked with oakum.

CHAPTER IX.

WATER-PROOFING AND PROTECTIVE TREATMENT.

A.—WATER-PROOFING.

78.—*General.*—The requirements for quality of concrete in Section 28 shall be strictly followed. Particular attention shall be given to workmanship.

79.—*Integral Compounds.*—Integral compounds shall not be used.

80.—*Membrane Water-Proofing.*—Membrane water-proofing shall be used in basements, pits, shafts, tunnels, bridge floors, retaining walls, and similar structures, where an added protection is desired.

81.—*Water-tight Joints.*—See Section 77.

B.—OIL-PROOFING.

82.—Concrete structures for containing light mineral oils, animal oils, certain vegetable oils, and other commercial liquids shall be given a special coating which shall be applied immediately after construction. Floors or other surfaces exposed to heavy concentrations of such oils or liquids shall be similarly protected. The treatment to be applied shall be approved by the Engineer.

C.—CONCRETE IN SEA WATER.

83.—*Proportions.*—Plain concrete in sea water or exposed directly along the sea coast shall contain not less than 1½ bbl. (6 bags) of Portland cement per cubic yard in place; concrete from 2 ft. below low water to 2 ft. above

high water, or from a plane below to a plane above wave action, shall be made of a mixture containing not less than 1¾ bbl. (7 bags) of Portland cement per cubic yard in place. Slag, broken brick, soft limestone, soft sandstone, or other porous or weak aggregates shall not be used.

84.—*Depositing.*—Concrete shall not be deposited under sea water, unless unavoidable, in which case it shall be placed in accordance with the methods described in Sections 48 to 51. Sea water shall not be allowed to come in contact with the concrete until it has hardened for at least 4 days. Concrete shall be placed in such a manner as to avoid horizontal or inclined seams or work planes. The placing of concrete between tides shall be a continuous operation, in accordance with the methods described in Section 42; where it is impossible to avoid seams or joints, proceed as in Section 43.

85.—*Protection.*—Metal reinforcement shall be placed at least 3 in. from any plane or curved surface, and at corners at least 4 in. from all adjacent surfaces. Metal chairs, supports, or ties shall not extend to the surface of the concrete. Where unusually severe conditions of abrasion are anticipated, the face of the concrete from 2 ft. below low water to 2 ft. above high water, or from a plane below to a plane above wave action, shall be protected by creosoted timber, dense vitrified shale brick, or stone of suitable quality, as designated on the plans.

86.—*Consistency.*—The consistency shall be such as to produce concrete which for mass work shall give a slump of not more than 2 in., and for reinforced concrete a slump of not more than 4 in.

D.—Concrete in Alkali Soils or Water.

87.—*Proportions.*—Concrete below the ground-line shall contain not less than 1¾ bbl. (7 bags) of Portland cement per cubic yard in place.

88.—*Consistency.*—The consistency of the concrete shall be such as to produce a slump of not more than 2 in., and for small members in which aggregates coarser than ⅜ in. cannot be used, a slump of not more than 6 in.

89.—*Placing.*—Concrete shall be placed in such a manner as to avoid horizontal, or inclined seams, or work planes; where this is impossible the requirements of Section 69 shall be followed.

90.—*Curing.*—Concrete shall be kept wet with fresh water for not less than 7 days following placing.

91.—*Protection.*—Metal reinforcement or other corrodible metal shall not be placed closer than 2 in. to the exposed faces of members exposed to alkali soil or water.

CHAPTER X.

Surface Finish.

92.—*General.*—Concrete to have exposed surfaces with specified finish shall be mixed, placed and worked to secure a uniform distribution of the aggregates, and insure uniform texture of surface.* Placing shall be continuous

* This is accomplished by uniform proportioning of ingredients and thorough mixing with the proper quantity of water; after placing, the concrete should be thoroughly rodded or forked to force the aggregate against the face forms and prevent the formation of voids.

throughout each distinct division of an area. Joint lines shall be located at indicated points. Voids which appear upon removal of the forms shall be drenched with water and be immediately filled with material of the same composition as that used in the surface, and smoothed with a wood spatula or float. Fins or offsets shall be neatly removed. The work shall be finished free from streaks.

93.—*Top Surfaces not Subject to Wear.*—Top surfaces not subject to wear shall be smoothed with a wood float and be kept wet for at least 7 days. Care shall be taken to avoid an excess of water in the concrete, and to drain off or otherwise promptly remove any water that comes to the surface. Dry cement, or a dry mixture of cement and sand, shall not be sprinkled directly on the surface.

A.—WEARING SURFACES.

94.—*One-Course Work.*—Aggregates for the wearing surface shall have a high resistance to abrasion. They shall be carefully screened and thoroughly washed. The least quantity of mixing water that will produce a dense concrete shall be used. The mix shall not be leaner than 1 part of Portland cement to $2\frac{1}{2}$ parts of aggregate. The surface shall be screeded even and finished with a wood float. Excess water shall be promptly drained off or otherwise removed. Over-troweling shall be avoided.

95.—*Two-Course Work.*—In two-course work the wearing surface shall be placed within $\frac{1}{2}$ hour after the base course.

If the wearing surface is required to be applied to a hardened base course, the latter shall be prepared by roughening with a pick or other effective tool, thoroughly drenching with water until saturated, and covered with a thin layer of neat cement immediately before the wearing surface is placed.

The finished wearing course in two-course work shall not be thinner than 1 in.

96.—*Curing.*—Concrete wearing surfaces constructed in accordance with Sections 94 and 95, shall be kept wet* for at least 10 days in the case of floors and 21 days in the case of roads and pavements.

97.†—*Terrazzo Finish.*—Terrazzo finish shall be constructed by mixing 1 part of Portland cement, $2\frac{1}{2}$ parts of crushed marble which will pass through a $\frac{1}{2}$-in. screen and is free from dust, and sufficient water to produce a dense concrete, which shall be spread on the base course and worked down to a thickness of 1 in. by patting or rolling and troweling.

The surface shall be kept wet for not less than 10 days and after thoroughly curing shall be rubbed to a plane surface with a stone or a surfacing machine. Hardened concrete to which a terrazzo finish is to be applied shall be prepared as prescribed in Section 95.

97.†—*Terrazzo Finish.*—Terrazzo finish shall be constructed by mixing 1 part of Portland cement, 2 parts of sand, and sufficient water to produce a

* Prevention of premature drying during the early hardening of concrete is essential to the development of high resistance to abrasion. The surface may be covered with a layer of burlap, earth, or sand, kept wet, or it may be divided into small areas by dikes and flooded with water to a depth of 2 or 3 in.

† The engineer should strike out one of the two Sections numbered 97.

plastic mortar, which shall be spread on the base course to a depth of 1 in. Crushed marble, which will pass through a $\frac{1}{4}$-in. screen and is free from dust, shall be sprinkled over the surface of the fresh mortar and pressed or rolled in.

The surface shall be kept wet for not less than 10 days and after thoroughly curing shall be rubbed to a plane surface with a stone or a surfacing machine. Hardened concrete to which a terrazzo finish is to be applied shall be prepared as prescribed in Section 95.

B.—Decorative Finishes.

98.—*Rubbed Finish.*—Concrete shall be wetted immediately after the forms are removed and rubbed even and smooth with a carborundum brick, or other abrasive, and to uniform appearance without applying any cement or other coating.

99.—*Scrubbed Finish.*—The face forms shall be removed as soon as the concrete has hardened sufficiently. Voids shall be immediately filled with mortar of the same composition as that used in the face. Fins and other unevennesses shall be rubbed off and the whole surface be scrubbed with fiber or wire brushes, using water freely, as the degree of hardness may require, until the aggregate is uniformly exposed; the surface shall then be rinsed with clean water. The corners shall be sharp and unbroken. If portions of the surface have become too hard to scrub in uniform relief, dilute hydrochloric acid (1 part of acid to 4 parts of water) may be used to facilitate scrubbing of hardened surfaces. The acid shall be thoroughly washed off with clean water.

100.—*Sand-Blast Finish.*—Immediately following removal of forms, voids shall be filled with mortar of the same composition as that used in the face, and allowed to harden. Unevennesses and form marks shall be removed by chipping or rubbing; the face shall then be cut with an air blast of hard sand with angular grains until the aggregate is in uniform relief.

101.—*Tooled Finish.*—The surface shall be permitted to become hard and dry before tooling. The cutting shall remove the entire skin and produce a uniform surface true to lines.

102.—*Sand-Floated Finish.*—The forms shall be removed before the surface has fully hardened; the surface shall be rubbed with a wooden float by a uniform circular motion, using fine sand until the resulting finish is even and uniform.

103.—*Colored Aggregate Finish.*—Colored or other special aggregate used for finish shall be exposed by scrubbing as provided in Section 99. Facing mortar of 1 part of Portland cement, $1\frac{1}{2}$ parts of sand, and 3 parts of screenings or pebbles shall be placed against the face forms to a thickness of about 1 in. sufficiently in advance of the body concrete to prevent the latter coming in contact with the form.

104.—*Colored Pigment Finish.*—Mineral pigment shall be thoroughly mixed dry with the Portland cement and fine aggregate; care shall be taken to secure a uniform tint throughout.

CHAPTER XI.

DESIGN.

A.—GENERAL ASSUMPTIONS.

105.—The design of reinforced concrete members under these specifications shall be based on the following assumptions:

(a) Calculations are made with reference to working stresses and safe loads rather than with reference to ultimate strength and ultimate loads.

(b) A plane section before bending remains plane after bending.

(c) The modulus of elasticity of concrete in compression is constant within the limits of working stresses; the distribution of compressive stress in beams is therefore rectilinear.

(d) The values for the modulus of elasticity of concrete in computations for the position of the neutral axis, for the resisting moment of beams, and for compression of concrete in columns, are as follows:

1. One-fortieth ($\frac{1}{40}$) that of steel, when the compressive strength of the concrete at 28 days is below 800 lb. per sq. in.
2. One-fifteenth ($\frac{1}{15}$) that of steel, when the compressive strength of the concrete at 28 days lies between 800 and 2 200 lb. per sq. in.
3. One-twelfth ($\frac{1}{12}$) that of steel, when the compressive strength of the concrete at 28 days lies between 2 200 and 2 900 lb. per sq. in.
4. One-tenth ($\frac{1}{10}$) that of steel, when the compressive strength of the concrete at 28 days is higher than 2 900 lb. per sq. in.
5. One-eighth ($\frac{1}{8}$) that of steel for calculating the deflection of reinforced concrete beams which are free to move longitudinally at the supports, and in which the tensile resistance of the concrete is neglected.

(e) In calculating the moment of resistance of reinforced concrete beams and slabs the tensile resistance of the concrete is neglected.

(f) The adhesion between the concrete and the metal reinforcement remains unbroken throughout the range of working stresses. Under compression the two materials are therefore stressed in proportion to their moduli of elasticity.

(g) Initial stress in the reinforcement due to contraction or expansion of the concrete is neglected, except in the design of reinforced concrete columns.

B.—FLEXURE OF RECTANGULAR REINFORCED CONCRETE BEAMS AND SLABS.

106.—*Flexure Formulas.*—Computations of flexure in rectangular reinforced concrete beams and slabs shall be based on the following formulas:

(a) *Reinforced for Tension Only:*

Position of neutral axis,

$$k = \sqrt{2\,p\,n + (p\,n)^2} - p\,n \quad\text{.................}(1)$$

Arm* of resisting couple,

$$j = 1 - \frac{k}{3} \quad\text{..........................}(2)$$

*For f_s = 16 000 to 18 000 lb. per sq. in. and f_c = 800 to 900 lb. per sq. in., j may be assumed as 0.86. For values of pn varying from 0.04 to 0.24, jk is approximately equal to $0.67\sqrt[3]{pn}$.

Compressive unit stress* in extreme fiber of concrete,

$$f_c = \frac{2\,M}{j\,k\,b\,d^2} = \frac{2\,p\,f_s}{k} \quad \ldots\ldots (3)$$

Tensile unit stress* in longitudinal reinforcement,

$$f_s = \frac{M}{A_s\,j\,d} = \frac{M}{p\,j\,b\,d^2} \quad \ldots\ldots (4)$$

Steel ratio for balanced reinforcement,

$$p = \frac{1}{2}\,\frac{1}{\dfrac{f_s}{f_c}\left(\dfrac{f_s}{n\,f_c} + 1\right)} \quad \ldots\ldots (5)$$

For formulas on shear and bond, see Sections 120 and 140.

(*b*) *Reinforced for Both Tension and Compression:*

Position of neutral axis,

$$k = \sqrt{2\,n\left(p + p'\,\frac{d'}{d}\right) + n^2\,(p + p')^2} - n\,(p + p') \quad \ldots\ldots (6)$$

Position of resultant compression,

$$z = \frac{\dfrac{1}{3}\,k^3\,d + 2\,p'\,n\,d'\left(k - \dfrac{d'}{d}\right)}{k^2 + 2\,p'\,n\left(k - \dfrac{d'}{d}\right)} \quad \ldots\ldots (7)$$

Arm of resisting couple,

$$j\,d = d - z \quad \ldots\ldots (8)$$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{6\,M}{b\,d^2\left[3\,k - k^2 + \dfrac{6\,p'\,n}{k}\left(k - \dfrac{d'}{d}\right)\left(1 - \dfrac{d'}{d}\right)\right]} \quad \ldots\ldots (9)$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{p\,j\,b\,d^2} = n\,f_c\,\frac{1 - k}{k} \quad \ldots\ldots (10)$$

Compressive unit stress in longitudinal reinforcement,

$$f'_s = n\,f_c\,\frac{k - \dfrac{d'}{d}}{k} \quad \ldots\ldots (11)$$

107.—*Notation.*—The symbols† used in Formulas 1 to 23 are defined as follows:

A_s = effective cross-sectional area of metal reinforcement in tension in beams;

b = width of rectangular beam, or width of flange of T-beam;

d = depth from compression surface of beam or slab to center of longitudinal tension reinforcement;

* For f_s = 16 000 to 18 000 lb. per sq. in. and f_c = 800 to 900 lb. per sq. in., j may be assumed as 0.86. For values of $p\,n$ varying from 0.04 to 0.24, $j\,k$ is approximately equal to 0.67 $\sqrt[3]{p\,n}$.

† For illustration of notation as applied to typical beams or slabs, see Figs. 1 and 2, Appendix II, p. 119.

d' = depth from compression surface of beam or slab to center of compression reinforcement;

f_c = compressive unit stress in extreme fiber of concrete;

f_s = tensile unit stress in longitudinal reinforcement;

f'_s = compressive unit stress in longitudinal reinforcement;

h = unsupported length of column;

I = moment of inertia of a section about the neutral axis for bending;

j = ratio of lever arm of resisting couple to depth, d;

k = ratio of depth of neutral axis to depth, d;

l = span length of beam or slab (generally distance from center to center of supports, see Section 108);

M = bending moment or moment of resistance in general;

$n = \frac{E_s}{E_c}$ = ratio of modulus of elasticity of steel to that of concrete;

p = ratio of effective area of tension reinforcement to effective area of concrete in beams $= \frac{A_s}{b\,d}$;

p' = ratio of effective area of compression reinforcement to effective area of concrete in beams;

w = uniformly distributed load per unit of length of beam or slab;

z = depth from compression surface of beam or slab to resultant of compressive stresses.

. . . .

108.—*Span Length.*—The span length, l, of freely supported beams and slabs shall be the distance between centers of the supports, but shall not exceed the clear span plus the depth of beam or slab. The span length for continuous or restrained beams built monolithically with supports shall be the clear distance between faces of supports. Where brackets having a width not less than the width of the beam, and making an angle of 45° or more with the axis of a restrained beam are built monolithic with the beam and support, the span shall be measured from the section where the combined depth of the beam and bracket is at least one-third more than the depth of the beam. Maximum negative moments are to be considered as existing at the ends of the span, as above defined. No portion of a bracket shall be considered as adding to the effective depth of the beam.

109.—*Moments in Freely Supported Beams of Equal Span.*—The following moments at critical sections of freely supported beams and slabs of equal spans carrying uniformly distributed loads shall be used:

(*a*) Maximum positive moment in beams and slabs of one span,

$$M = \frac{w\,l^2}{8} \qquad (12)$$

(*b*) Center of slabs and beams continuous for two spans only,

1.—Positive moment at the center,

$$M = \frac{w\,l^2}{10} \qquad (13)$$

2.—Maximum negative moment,

$$M = \frac{w\, l^2}{8} \quad \text{(14)}$$

(*c*) Slabs and beams continuous for more than two spans,

1.—Center and supports of interior spans,

$$M = \frac{w\, l^2}{12} \quad \text{(15)}$$

2.—Center and interior support of end spans,

$$M = \frac{w\, l^2}{10} \quad \text{(16)}$$

(*d*) Negative moment at the supports of slab or beam built into brick or masonry walls in a manner that develops partial end restraint,

$$M = \text{not less than } \frac{w\, l^2}{16} \quad \text{(17)}$$

110.—*Moments in Beams Monolithic with Supports.*—The following moments at the critical sections of beams or slabs of equal spans cast monolithic with columns or similar supports, and carrying uniformly distributed loads shall be used:

(*a*) Supports of intermediate spans,

$$M = \frac{w\, l^2}{12} \quad \text{(18)}$$

(*b*) Center of intermediate spans,

$$M = \frac{w\, l^2}{16} \quad \text{(19)}$$

(*c*) Beams in which $\frac{I}{l}$ is less than twice the sum of the value of $\frac{I}{h}$ for the exterior columns above and below, which are built into the beam,

1.—Center and first interior support,

$$M = \frac{w\, l^2}{12} \quad \text{(20)}$$

2.—Exterior supports,

$$M = \frac{w\, l^2}{12} \quad \text{(21)}$$

(*d*) Beams in which $\frac{I}{l}$ is equal to, or greater than, twice the sum of the values of $\frac{I}{h}$ for the exterior columns above and below which are built into the beam,

1.—Center of span and at first interior support of end span,

$$M = \frac{w\, l^2}{10} \quad \text{(22)}$$

2.—Exterior support,

$$M = \frac{w\, l^2}{16} \quad \text{(23)}$$

111.—*Moment Coefficients of Continuous Beams.*—Continuous beams with unequal spans, whether freely supported or cast monolithic with columns, shall be analyzed to determine the actual moments under the given conditions of loading and restraint. Provision shall be made for negative moment occurring in short spans adjacent to longer spans when the latter only are loaded.

C.—Flexure of Reinforced Concrete T-Beams.

112.—*Flexure Formulas.*—Computations of flexure in reinforced concrete T-beams shall be based on the following formulas:

(*a*) *Neutral Axis in the Flange:*

Use formulas for rectangular beams and slabs in Section 106.

(*b*) *Neutral Axis Below the Flange*:*

Position of neutral axis,

$$k\,d = \frac{2\,n\,d\,A_s + b\,t^2}{2\,n\,A_s + 2\,b\,t} \qquad \text{(24)}$$

Position of resultant compression,

$$z = \left(\frac{3\,k\,d - 2\,t}{2\,k\,d - t}\right)\frac{t}{3} \qquad \text{(25)}$$

Arm of resisting couple,

$$j\,d = d - z \qquad \text{(26)}$$

Compressive unit stress in extreme fiber of concrete,

$$f_c = \frac{M\,k\,d}{b\,t\left(k\,d - \frac{1}{2}\,t\right)j\,d} = \frac{f_s}{n}\left(\frac{k}{l - k}\right) \qquad \text{(27)}$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{A_s\,j\,d} \qquad \text{(28)}$$

Formulas 24, 25, 26, 27, and 28 neglect compression in the stem.†

* For approximate results the formulas for rectangular beams, Section 106, may be used.

† The following formulas take into account the compression in the stem; they are recommended where the flange is small compared with the stem:

Position of neutral axis,

$$k\,d = \sqrt{\frac{2\,n\,d\,A_s + (b - b')\,t^2}{b'} + \left(\frac{n\,A_s + (b - b')\,t}{b'}\right)^2} - \frac{n\,A_s + (b - b')\,t}{b'} \qquad \text{(24a)}$$

Position of resultant compression,

$$z = \frac{\left(k\,d\,t^2 - \frac{2}{3}\,t^3\right)b + \left[(k\,d - t)^2\left(t + \frac{1}{3}(k\,d - t)\right)\right]b'}{t\,(2\,k\,d - t)\,b + (k\,d - t)^2\,b'} \qquad \text{(25a)}$$

Arm of resisting couple (See footnote, Section 106),

$$j\,d = d - z \qquad \text{(26a)}$$

Compressive unit stress in extreme fibre of concrete,

$$f_c = \frac{2\,M\,k\,d}{[(2\,k\,d - t)\,b\,t + (k\,d - t)^2\,b']\,j\,d} \qquad \text{(27a)}$$

Tensile unit stress in longitudinal reinforcement,

$$f_s = \frac{M}{A_s\,j\,d} \qquad \text{(28a)}$$

113.—*Notation.*—The symbols* used in Formulas 24 to 28 are defined in Section 107, except as follows:

b' = width of stem of T-beam;

t = thickness of flanges of T-beam.

114.—*Flange Width.*—Effective and adequate bond and shear resistance shall be provided in beam-and-slab construction at the junction of the beam and slab; the slab shall be built and considered an integral part of the beam; the effective flange width shall not exceed one-fourth of the span length of the beam, and its overhanging width on either side of the web shall not exceed 8 times the thickness of the slab nor one-half the clear distance to the next beam.

115.—*Flange Length.*—The unsupported length of the compression flange of a T-beam shall not exceed 36 times the least width of the beam.

116.—*Transverse Reinforcement.*—Where the principal slab reinforcement is parallel to the beam, transverse reinforcement, not less in amount than 0.3% of the sectional area of the slab, shall be provided in the top of the slab, and shall extend over the beam and into the slab not less than two-thirds of the effective flange overhang. The spacing of the bars shall not exceed 18 in.

117.—*Compressive Stress in Supports.*—Provision shall be made for the compressive stress at the support in continuous T-beam construction.

118.—*Shear.*—The flange of the slab shall not be considered as effective in computing the shear and diagonal tension resistance of T-beams.

119.—*Isolated Beams.*—Isolated beams in which the T-form is used only for the purpose of providing additional compression area, shall have a flange thickness not less than one-half the width of the web, and a total flange width not more than 4 times the web thickness.

D.—Diagonal Tension and Shear.

(*a*) *Formulas and Notation.*

120.—*Formulas.*—Diagonal tension and shear in reinforced concrete beams shall be calculated by the following formulas:

Shearing unit stress†,

$$v = \frac{V}{b\,j\,d} \qquad \text{(29)}$$

Stress† in vertical web reinforcement,

$$f_v = \frac{V' s}{A_v\,j\,d} \qquad \text{(30)}$$

121.—*Notation.*—The symbols used in Formulas 29 to 36 are defined in Section 107, except as follows:

a = spacing of web reinforcement bars measured perpendicular to their direction;

A_v = total area of web reinforcement in tension within a distance of a (a_1, a_2, a_3, etc.), or the total area of all bars bent up in any one plane;

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* For illustration of certain symbols as applied to typical T-beams, see Fig. 3, Appendix II, p. 119.

† Approximate results may be secured by assuming j = 0.875.

α = angle between web bars and longitudinal bars;
f_v = tensile unit stress in web reinforcement;
o = perimeter of bar;
Σ_o = sum of perimeters of bars in one set;
r = ratio of cross-sectional area of negative reinforcement which crosses entirely over the column capital of a flat slab or over the dropped panel, to the total cross-sectional area of the negative reinforcement in the two column strips;
s = spacing of web members, measured at the neutral axis and in the direction of longitudinal axis of the beam;
u = bond stress per unit of area of surface of bar;
v = shearing unit stress;
V = total shear;
V' = external shear on any section after deducting that carried by the concrete.

(b) Beams Without Web Reinforcement.

122.—*Bars Not Anchored.*—The shearing unit stress in beams in which the longitudinal reinforcement is designed to meet all moment requirements, but without special anchorage, shall not exceed $0.02f'_c$, but in no case shall it exceed 40 lb. per sq. in. Adequate reinforcement shall be provided at all sections where negative moment occurs in beams continuous over supports or built into walls or columns at their ends. (For typical design see Fig. 4*).

123.—*Bars Anchored.*—The shearing unit stress in beams in which longitudinal reinforcement is anchored by means of hooked ends or otherwise, as specified in Section 130, shall not exceed $0.03f'_c$. Adequate reinforcement for both positive and negative moment shall be provided at all sections where maximum moment exists. (For typical design, see Fig. 5*).

(c) Beams With Web Reinforcement.

124.—*With Web Reinforcement.*—When the shearing unit stress calculated by Formula 29 exceeds the values specified in Sections 122 and 123, web reinforcement shall be provided by one or more of the following methods:

(*a*) Series of vertical stirrups or web bars;
(*b*) Series of inclined stirrups or web bars;
(*c*) Series of bent-up longitudinal bars;
(*d*) Longitudinal bars bent up in a single plane.

Provision against bond failure of the web reinforcement shall be as specified in Section 131. (For typical designs, see Figs. 6 and 7*; for typical detail of anchorage of longitudinal bars and vertical stirrups, see Fig. 8*).

125.—*Web or Bent-Up Bars.*—Where web reinforcement is present and where longitudinal reinforcement is provided to meet all moment requirements, the concrete may be assumed to carry a shearing unit stress not greater than $0.02f'_c$ and not greater in any case than 40 lb. per sq. in. In the

* Appendix II, pp. 120–121.

case where a series of web bars or bent-up longitudinal bars is used, the web reinforcement shall be designed according to the formula:

$$A_v = \frac{V' \, a}{f_v j \, d} = \frac{V' \, s \sin \alpha}{f_v j \, d} \qquad (31)$$

(For typical design, see Fig. 9*).

126.—*Bars Bent Up in Single Plane.*—Where the web reinforcement consists of bars bent up in a single plane at an angle so as to reinforce all sections of the beam in which the shearing unit stress on the web concrete exceeds $0.02f'_c$, the concrete may be assumed to take a shearing unit stress not greater than $0.02f'_c$, and not greater than 40 lb. per sq. in.; the remainder of the shear shall be carried by the bent-up bars designed according to the formula:

$$A_v = \frac{V'}{f_v \sin \alpha} \qquad (32)$$

In case the web reinforcement consists solely of bent bars, the first bent bar shall bend downward from the plane of the upper reinforcement at the plane of the edge of the support or between that plane and the center of the support. (For typical design, see Fig. 10*).

127.—*Combined Web Reinforcement.*—Where two or more types of web reinforcement are used in conjunction, the total shearing resistance of the beam shall be taken as the sum of the shearing resistances as computed for the various types separately.†

128.—*Maximum Shearing Unit Stress.*—Where there is no special mechanical anchorage of the longitudinal reinforcement, the shearing unit stress shall not exceed $0.06f'_c$, irrespective of the web reinforcement used.

129.—*Special Mechanical Anchorage.*—Where special mechanical anchorage of the longitudinal reinforcement as prescribed in Section 130 is provided, the shearing unit stress as computed by Formula 29, may be greater than $0.06f'_c$, but in no case shall it exceed $0.12f'_c$.‡ In this case the concrete may be assumed to take a shearing unit stress of not more than $0.025f'_c$, but not more than 50 lb. per sq. in.

130.—*Anchorage of Longitudinal Reinforcement.*—Special mechanical anchorage of the longitudinal reinforcement for positive moment may consist of carrying the bars a sufficient distance beyond the point of inflection of restrained or continuous members to develop by bond between the point of inflection and the end of the bar a tensile stress equal to one-third the safe working stress in the reinforcement. If such a bar is straight, it shall extend to within 1 in. of the center of the support, or in the case of wide supports shall extend not less than 12 in. beyond the face of the support. Special mechanical anchorage may also be secured by bending the end of the bar over the support in a full semi-circle to a diameter not less than 8 times

* Appendix II, pp. 121–122.

† In such computation the shearing value of the concrete in the web shall be included once only.

‡ The limit, $0.12f'_c$, is based on the ultimate bearing unit stress of $0.5f'_c$ at which beams reinforced with vertical stirrups fail due to diagonal compression in the webs. A higher value than $0.12f'_c$ may be permitted in beams with inclined web reinforcement, but it is not thought necessary to allow such higher limit to meet the needs of design practice.

the diameter of the bar, the total length of the bend being not less than 16 diameters of the bar. Any other mechanical device that secures the end of the bar over the support against slipping without stressing the concrete in excess of $0.5f'_c$ in local compression may be used, provided such device does not tend to split the concrete. Negative reinforcement shall be thoroughly anchored at or across the support, or shall extend into the span a sufficient distance to develop by bond the tensile stresses due to negative moment. In the case of freely supported ends of continuous beams, special mechanical anchorage shall be provided, which is capable of developing at the end of the span a tensile stress which is not less than one-third of the safe tensile stress of the bar at the point of maximum moment. (For illustrative design, see Fig. 11*).

131.—*Anchorage of Web Reinforcement.*—Anchorage of the web reinforcement shall be by one of the following methods:

(*a*) Continuity of the web bar with the longitudinal bar;

(*b*) Carrying the web bar around at least two sides of a longitudinal bar at both ends of the web bar; or

(*c*) Carrying the web bar about at least two sides of a longitudinal bar at one end and making a semi-circular hook at the other end which has a diameter equal to that of the web bar.

In all cases the bent ends of web bars shall extend at least 8 diameters below or above the point of extreme height or depth of the web bar. In case the end anchorage of the web member is not in bearing on other reinforcement, the anchorage shall be such as to engage an adequate amount of concrete to prevent the bar from pulling off a portion of the concrete. In all cases the stirrups shall be carried as close to the upper and lower surfaces as fire-proofing requirements will permit. (For typical designs, see Fig. 8* and 12*).

132.—*Size of Web Bars.*—The size of web reinforcing bars which are neither a part of the longitudinal bars nor welded thereto shall be such that not less than two-fifths of the allowable tensile stress in the bar may be developed by bond stresses in a length of bar equal to $0.4d$.† The remainder of the tensile stress in the bar shall be provided for by adequate end anchorage, as specified in Section 131.

133.—*Breadth of Beams in Shear.*—Shearing unit stress shall be computed on the full width of rectangular beams, on the width of the stem of T-beams, and on the thickness of the web in beams of I-section.

134.—*Shear in Beam-and-Tile Construction.*—The shearing stress in tile-and-concrete-beam construction shall not exceed that in beams or slabs with similar reinforcement. The width of the effective section for shear, as governing diagonal tension, shall be taken as the thickness of the concrete web plus one-half the thickness of the vertical webs of the tile. (For typical design, see Fig. 13*).

* Appendix II, pp. 121–122.

† This condition is satisfied for plain round stirrups when the diameter of the bar does not exceed $\frac{d}{50}$

135.—*Spacing of Web Reinforcement.*—The spacing, a, of web reinforcement bars shall be measured perpendicular to their direction and in a plane parallel to the longitudinal axis of the beam. The spacing shall not exceed $\frac{3}{4}d$ in any case where web reinforcement is necessary. Where vertical stirrups are used, or where inclined web bars make an angle more than 60° with the horizontal, the spacing shall not exceed $\frac{1}{2}d$. Where the shearing unit stress exceeds $0.06f'_c$, the spacing of the web reinforcement shall not exceed $\frac{1}{2}d$ in any case, nor $\frac{1}{3}d$ for vertical stirrups or web reinforcement making an angle more than 60° with the horizontal. The first shear reinforcement member shall cross the neutral axis of the member at a distance from the face of the support, measured along the axis of the beam, not greater than $\frac{1}{4}d$, nor greater than the spacing of web members as determined for a section taken at the edge of the support. Web members may be placed at any angle between 20° and 90° with the longitudinal bars, provided that if inclined they shall be inclined in such a manner as to resist the tensile stress in the web.

(*d*) *Flat Slabs.*

136.—*Shearing Stress.*—The shearing unit shearing stress shall not exceed the value of v in the formula,

$$v = 0.02\, f'_c\, (1 + r) \qquad (33)$$

nor in any case shall it exceed $0.03\, f'_c$.

The unit shearing stress shall be computed on

(*a*) A vertical section which has a depth, in inches, of $\frac{7}{8}\,(t_1 - 1\frac{1}{2})$, and which lies at a distance, in inches, of $t_1 - 1\frac{1}{2}$ from the edge of the column capital; and

(*b*) A vertical section which has a depth, in inches, of $\frac{7}{8}\,(t_2 - 1\frac{1}{2})$, and which lies at a distance, in inches, of $t_2 - 1\frac{1}{2}$ from the edge of the dropped panel.

In no case shall r be less than 0.25. Where the shearing stress on section (*a*) is being considered, r shall be taken as the proportional amount of reinforcement crossing the column capital; where the shearing stress at section (*b*) is being considered, r shall be taken as the proportional amount of reinforcement crossing entirely over the dropped panel. (For typical flat slab and designation of principal design sections, see Figs. 14 and 15*).

(*e*) *Footings.*

137.—*Shear and Diagonal Tension in Footings.*—The shearing stress shall be computed by Formula 29. When so computed the stress on the critical section defined below, or on sections outside of the critical section, shall not exceed $0.02\, f'_c$ for footings with straight reinforcement bars, nor $0.03\, f'_c$ for footings in which the reinforcement bars are anchored at both ends by adequate hooks or otherwise, as specified in Section 130.

138.—*Critical Section of Soil Footings.*—The critical section for diagonal tension in footings bearing directly on the soil shall be taken on a vertical

* See Appendix II, p. 123.

section through the perimeter of the lower base of a frustum of a cone or pyramid which has a base angle of 45°, and has for its top the base of the column or pedestal and for its lower base the plane of the center of the longitudinal reinforcement.

139.—*Critical Section for Pile Footings.*—The critical section for diagonal tension in footings bearing on piles shall be taken on a vertical section at the inner edge of the first row of piles entirely outside a section midway between the face of the column or pedestal and the section described in Section 138 for soil footings, but in no case outside of the section described in Section 138. The critical section for piles not grouped in rows shall be taken midway between the face of the column and the perimeter of the base of the frustum described in Section 138.

E.—Bond.

140.—*Formula.*—Bond between concrete and reinforcement bars in reinforced concrete beams and slabs shall be computed by the formula:

$$u = \frac{V}{j\,d\,\Sigma\,o} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

141.—*Working Stress.*—Unless otherwise specified, the reinforcement shall be so proportioned that the bond stress between the metal and the concrete shall not exceed the following:

(*a*) Plain bars,

$$u = 0.04\,f'_c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

(*b*) Deformed bars, meeting the requirements of Section 23,

$$u = 0.05\,f'_c \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

142.—*Bond in Footings.*—The bond stress on a section of a footing shall be computed by Formula 34. Only the bars counted as effective in bending shall be considered in computing the number of bars crossing a section. The bond stress computed in this manner on sections at the face of the column or outside the column shall not exceed the value specified in Section 141. Special investigation shall be made of bond stresses in footings with stepped or sloping upper surface; maximum stresses may occur at sections near the edges of the footings.

143.—*Reinforcement in Two or More Directions.*—The permissible bond stress given by Formulas 35 and 36 for footings and similar members where reinforcement is required in more than one direction shall be reduced as follows:

(*a*) For two-way reinforcement............................25 per cent.
(*b*) For each additional direction..........................10 per cent.

144.—*Anchored Bars.*—The bond stresses for bars adequately anchored at both ends by hooks or otherwise, as provided in Section 130, may be 1½ times the values specified in Section 141. Hooks in footings shall be effectively positioned to insure that they engage a mass of concrete above the plane of the reinforcement.

F.—Flat Slabs.*

145.—*Moments in Interior Panels.*—The symbols used in Formulas 37 to 42 are defined in Section 107 except as indicated in Sections 145, 148 and 158.

In flat slabs in which the ratio of reinforcement for negative moment in the column strip is not greater than 0.01, the numerical sum of the positive and negative moments in the direction of either side of the panel shall be taken as not less than

$$M_0 = 0.09\, W\, l \left(1 - \frac{2}{3}\frac{c}{l}\right)^2 \dots\dots\dots\dots\dots\dots (37)$$

Where M_0 = sum of positive and negative bending moments in either rectangular direction at the principal design sections of a panel of a flat slab;

c = base diameter of the largest right circular cone which lies entirely within the column (including the capital) whose vertex angle is 90° and whose base is 1½ in. below the bottom of the slab or the bottom of the dropped panel (See Fig. 14†);

l = span length‡ of flat slab, center to center of columns in the rectangular direction, in which moments are considered; and

W = total dead and live load uniformly distributed over a single panel area.

146.—*Principal Design Sections.*—The principal design sections for critical moments in flat slabs subjected to uniform load shall be taken as follows:

(*a*) Negative moment in middle strip: Extending in a rectangular direction from a point on the edge of panel $\frac{l_1}{4}$ from column center a distance $\frac{l_1}{2}$ toward the center of adjacent column on the same panel edge.

(*b*) Negative moment in column strip: Extending in a rectangular direction along the edge of the panel from a point $\frac{l_1}{4}$ from the center of the column to a point $\frac{c}{2}$ from the center of the same column and thence one-quarter circumference about the column center to the adjacent edge of the panel.

(*c*) Positive moment in middle strip: Extending in a rectangular direction from the center of one edge of a middle strip a distance $\frac{l_1}{2}$ to the center of the other edge of the same strip.

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* The requirements for flat slabs in Sections 145 to 162, inclusive, apply to two-way and four-way systems of reinforcement. The Committee is not prepared at this time to submit requirements covering other types of flat slabs.

† Appendix II, p. 123.

‡ The column strip and the middle strip to be used when considering moments in the direction of the dimension l are located and dimensioned as shown in Fig. 15. The dimension l_1 does not always represent the short length of the panel. When moments in the direction of the shorter panel length are considered, the dimensions l and l_1 are to be interchanged and the strips corresponding to those shown in Fig. 15 but extending in the direction of the shorter panel length are to be considered.

(*d*) Positive moment in column strip: Extending in a rectangular direction from the center of one edge of a column strip a distance $\frac{l_1}{4}$ to the center of the other edge of the same strip.

147.—*Moments in Principal Design Sections.*—The moments in the principal design sections shall be those given in Table 3, except as follows:

(*a*) The sum of the maximum negative moments in the two column strips may be greater or less than the values given in Table 3 by not more than 0.03 M_0.

(*b*) The maximum negative moment and the maximum positive moments in the middle strip and the sum of the maximum positive moments in the two column strips may each be greater or less than the values given in Table 3 by not more than 0.01 M_0.

TABLE 3.—MOMENTS TO BE USED IN DESIGN OF FLAT SLABS.

Strip.	Flat Slabs without Dropped Panels.		Flat Slabs with Dropped Panels.	
	Negative.	Positive.	Negative.	Positive.
Slabs with Two-Way Reinforcement.				
Column strip	0.23 M_0	0.11 M_0	0.25 M_0	0.10 M_0
Two-column strips	0.46 M_0	0.22 M_0	0.50 M_0	0.20 M_0
Middle strip	0.16 M_0	0.16 M_0	0.15 M_0	0.15 M_0
Slabs with Four-Way Reinforcement.				
Column strip	0.25 M_0	0.10 M_0	0.27 M_0	0.095 M_0
Two-column strips	0.50 M_0	0.20 M_0	0.54 M_0	0.190 M_0
Middle strip	0.10 M_0	0.20 M_0	0.08 M_0	0.190 M_0

148.—*Thickness of Flat Slabs and Dropped Panels.*—The total thickness,* t_1, of the dropped panel, in inches, or of the slab if a dropped panel is not used, shall be not less than:

$$t_1 = 0.0382 \left(1 - 1.44 \frac{c}{l}\right) l \sqrt{R\, w' \frac{l_1}{b_1}} + 1\tfrac{1}{2} \dots\dots\dots\dots (38)\dagger$$

where R = ratio of negative moment in the two-column strips to M_0; and w' = uniformly distributed dead and live load per unit of area of floor.

For slabs with dropped panels, the total thickness,* in inches, at points away from the dropped panel shall be not less than:

$$t_2 = 0.02\, l \sqrt{w'} + 1 \dots\dots\dots\dots (39)$$

* The thickness will be in inches regardless of whether l and w' are in feet and pounds per square foot or in inches and pounds per square inch.

† The values of R used in this formula are the coefficients of M_0 for negative moment in the two-column strips in Table 3, Section 147.

The slab thickness, t_1 or t_2, shall in no case be less than $\frac{l}{32}$ for floor-slabs, and not less than $\frac{l}{40}$ for roof slabs. In determining minimum thickness by Formulas 38 and 39, the value of l shall be the panel length center to center of the columns on the long side of the panel, l_1 shall be the panel length on the short side of the panel, and b_1 shall be the width or diameter of dropped panel in the direction of l_1, except that in a slab without dropped panel, b_1 shall be $0.5l_1$,

149.—*Minimum Dimensions of Dropped Panels.*—The dropped panel shall have a length or diameter in each rectangular direction of not less than one-third the panel length in that direction, and a thickness not greater than $1.5t_2$.

150.—*Wall and Other Irregular Panels.*—In wall panels and other panels in which the slab is discontinuous at the edge of the panel, the maximum negative moment one panel length away from the discontinuous edge and the maximum positive moment between shall be taken as follows:

(*a*) Column strip perpendicular to the wall or discontinuous edge, 15% greater than that given in Table 3.

(*b*) Middle strip perpendicular to wall or discontinuous edge, 30% greater than that given in Table 3.

In these strips the bars used for positive moments perpendicular to the discontinuous edge shall extend to the exterior edge of the panel at which the slab is discontinuous.

151.—*Panels with Wall Beams.*—In panels having a marginal beam on one edge or on each of two adjacent edges, the beam shall be designed to carry the load superimposed directly upon it. If the beam has a depth greater than the thickness of the dropped panel into which it frames, the beam shall be designed to carry, in addition to the load superimposed upon it, at least one-quarter of the distributed load for which the adjacent panel or panels are designed, and each column strip adjacent to and parallel with the beam shall be designed to resist a moment at least one-half as great as that specified in Table 3 for a column strip.* If the beam used has a depth less than the thickness of the dropped panel into which it frames, each column strip adjacent to and parallel with the beam shall be designed to resist the moments specified in Table 3 for a column strip. Where there are beams on two opposite edges of the panel, the slab and the beam shall be designed as though all the load was carried to the beam.

152.—*Discontinuous Panels.*—The negative moments on sections at and parallel to the wall, or discontinuous edge of an interior panel, shall be determined by the conditions of restraint.†

153.—*Flat Slabs on Bearing Walls.*—Where there is a beam or a bearing wall on the center line of columns in the interior portion of a continuous flat slab, the negative moment at the beam or wall line in the middle strip perpendicular to the beam or wall shall be taken as 30% greater than the

* In wall columns, brackets are sometimes substituted for capitals or other changes are made in the design of the capital. Attention is directed to the necessity for taking into account the change in the value of c in the moment formula for such cases.

† The Committee is not prepared to make a more definite recommendation at this time.

moment specified in Table 3 for a middle strip. The column strip adjacent to and lying on either side of the beam or wall shall be designed to resist a moment at least one-half of that specified in Table 3 for a column strip.

154.—*Point of Inflection.*—The point of inflection in any line parallel to a panel edge in interior panels of symmetrical slabs without dropped panels shall be assumed to be at a distance from the center of the span equal to $\frac{3}{10}$ of the distance between the two sections of critical negative moment at opposite ends of the line; for slabs having dropped panels, the coefficient shall be $\frac{1}{4}$.

155.—*Reinforcement.*—The reinforcement bars which cross any section and which fulfill the requirements given in Section 156 may be considered as effective in resisting the moment at the section. The sectional area of a bar multiplied by the cosine of the angle between the direction of the axis of the bar and any other direction may be considered effective as reinforcement in that direction.

156.—*Arrangement of Reinforcement.*—The design shall include adequate provision for securing the reinforcement in place so as to take not only the critical moments, but the moments at intermediate sections. All bars in rectangular or diagonal directions shall extend on each side of a section of critical moment, either positive or negative, to points at least 20 diameters beyond the point of inflection as specified in Section 154. In addition to this provision, bars in diagonal directions used as reinforcement for negative moment shall extend on each side of a line drawn through the column center at right angles to the direction of the band at least a distance of 0.35 of the panel length, and bars in diagonal directions used as reinforcement for positive moment shall extend on each side of a diagonal through the center of the panel at least a distance equal to 0.35 of the panel length; no splice by lapping shall be permitted at or near regions of maximum stress except as just described. At least two-thirds of all bars in each direction shall be of such length and shall be so placed as to provide reinforcement at two sections of critical negative moment and at the intermediate section of critical positive moment. Continuous bars shall not all be bent up at the same point of their length, but the zone in which this bending occurs shall extend on each side of the assumed point of inflection, and shall cover a width of at least $\frac{1}{15}$ of the panel length. Mere sagging of the bars shall not be permitted. In four-way reinforcement the position of the bars in both diagonal and rectangular directions may be considered in determining whether the width of zone of bending is sufficient.

157.—*Reinforcement at Construction Joints.*—See Section 73.

158.—*Tensile Stress in Reinforcement.*—The following formula shall be used in computing the tensile stress, f_s, in the reinforcement in flat slabs; the stress so computed shall not at any of the principal design sections exceed the values specified in Section 205:

$$f_s = \frac{R\, M_0}{A_s\, j\, d} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (40)$$

Where RM_0 = moment specified in Section 147 for two-column strips or for one middle strip; and

A_s = effective cross-sectional area of the reinforcement which crosses any of the principal design sections and which meets the requirements of Section 156.

159.—*Compressive Stress in Reinforcement.*—The following formulas shall be used in computing maximum compressive stress in the concrete in flat slabs; but the stress so computed shall not exceed $0.4f'_c$:

(*a*) Compression due to negative moment, RM_0, in the two-column strips,

$$f_c = \frac{3.5\, R\, M_0}{k\, j\, b_1\, d^2}\left(1 - 1.2\,\frac{c}{l}\right) \qquad (41)$$

where b_1 is as specified in Section 148.

(*b*) Compression due to positive moment, RM_0, in the two-column strips, or negative or positive moment in the middle strip,

$$f_c = \frac{6\, R\, M_0}{k\, j\, l_1\, d^2} \qquad (42)$$

160.—*Shearing Stress.*—See Section 136.

161.—*Unusual Panels.*—The moment coefficients, moment distribution, and slab thicknesses specified herein are for slabs which have three or more rows of panels in each direction, and in which the panels are approximately uniform in size. For structures having a width of one or two panels, and also for slabs having panels of markedly different sizes, an analysis shall be made of the moments developed in both slab and columns, and the values given herein modified accordingly. Slabs with paneled ceiling or with depressed paneling in the floor shall be considered as coming under the requirements herein given.

162.—*Bending Moments in Columns.*—See Section 173.

G.—Reinforced Concrete Columns.

163.—*Limiting Dimensions.*—The provisions in the following sections for the carrying capacity of reinforced columns are based on the assumption of a short column. Where the unsupported length is greater than 40 times the least radius of gyration ($40R$), the carrying capacity of the column shall be determined by Formula 48 for slender columns. Principal columns in buildings shall have a width of not less than 12 in. Posts that are not continuous from story to story shall have a width of not less than 6 in.

164.—*Unsupported Length.*—The unsupported length of a column in flat slab construction shall be taken as the clear distance between the under side of the capital and the top of the floor-slab below. For beam-and-slab construction the unsupported length of a column shall be taken as the clear distance between the under side of the shallowest beam framing into it and the top of the floor-slab below; where beams run in one direction only the clear distance between floor-slabs shall be used. For columns supported laterally by struts or beams only, two struts shall be considered an adequate support, provided the angle between the two planes formed by the axis of the

column with the axis of each strut is not less than 75°, nor more than 105°. The unsupported length for this condition shall be considered the clear distance between struts. When haunches are used at the junction of beams or struts with columns, the clear distance between supports may be considered as reduced by two-thirds of the depth of the haunch.

165.—*Safe Load on Spiral Columns.*—The symbols used in Formulas 43 to 50 are defined in Section 107, except as indicated in Sections 165, 167, 170, 172, 180 and 188. The safe axial load on columns reinforced with longitudinal bars and closely spaced spirals enclosing a circular core shall be determined by the following formula:

$$P = A_c f_c + n f_c p A \qquad (43)$$

where A = area of the concrete core enclosed within the spiral;

P = total safe axial load on column whose $\frac{h}{R}$ is less than 40;

p = ratio of effective area of longitudinal reinforcement to area of the concrete core; and

$A_c = A\ (1 - p)$ = net area of concrete core.

The allowable value of f_c for use in this type of column shall be determined by the following formula:

$$f_c = 300 + (0.10 + 4\,p)\, f'_c \qquad (44)$$

The longitudinal reinforcement shall consist of at least six bars of minimum diameter of ½ in., and its effective cross-sectional area shall not be less than 1% nor more than 5% of that of the enclosed core.

166.—*Spiral Reinforcement.*—The spiral reinforcement shall be not less in amount than one-fourth the volume of the longitudinal reinforcement. It shall consist of evenly spaced continuous spirals held firmly in place and true to line by at least three vertical spacer bars. The spacing of the spirals shall not be greater than one-sixth of the diameter of the core and in no case more than 3 in. The lateral reinforcement shall meet the requirements of the "Tentative Specifications for Cold-Drawn Steel Wire for Concrete Reinforcement" (Serial Designation: A9—21) of the American Society for Testing Materials. (Appendix VII)*. Reinforcement shall be protected everywhere by a covering of concrete cast monolithic with the core, which shall have a minimum thickness of 1½ in. in square columns and 2 in. in round or octagonal columns.

167.—*Safe Load on Columns with Lateral Ties.*—The safe axial load on columns reinforced with longitudinal bars and separate lateral ties shall be determined by the following formula:

$$P = A'_c f_c + A_s n f_c \qquad (45)$$

where A'_c = net area of concrete in the column (total column area minus steel area); and

A_s = effective cross-sectional area of longitudinal reinforcement.

The value of f_c for this type of column shall not exceed $0.20 f'_c$. The amount of longitudinal reinforcement considered in the calculations shall not be more

* Not reproduced.

than 2% nor less than 0.5% of the total area of the column. The longitudinal reinforcement shall consist of not less than four bars of minimum diameter of ½ in., placed with clear distance from the face of the column not less than 2 in.

168.—*Lateral Ties.*—Lateral ties shall be not less than ¼ in. in diameter, spaced not farther than 8 in. apart.

169.—*Bending Stress in Columns.*—Reinforced concrete columns subjected to bending stresses shall be treated as follows:

(*a*) *Spiral Column.*—The compressive unit stress on the concrete within the core area under combined axial load and bending shall not exceed by more than 20% the value given for axial load by Formula 44.

(*b*) *Columns with Lateral Ties.*—Additional longitudinal reinforcement not to exceed 2% shall be used if required and the compressive unit stress on the concrete under combined axial load and bending may be increased to $0.30f'_c$.

Tension due to bending in the longitudinal reinforcement shall not exceed 16 000 lb. per sq. in.

170.—*Composite Columns.*—The safe carrying capacity of composite columns in which a structural steel or cast-iron column is thoroughly encased in a spirally reinforced concrete core shall be based on a certain unit stress for the steel or cast-iron core, plus a unit stress of $0.25f'_c$ on the area within the spiral core. The unit compressive stress on the steel section shall be determined by the formula:

$$f_r = 18\,000 - 70\,\frac{h}{R} \quad \text{.......................(46)}$$

but shall not exceed 16 000 lb. per sq. in. The unit stress on the cast-iron section shall be determined by the formula:

$$f_r = 12\,000 - 60\,\frac{h}{R} \quad \text{.......................(47)}$$

but shall not exceed 10 000 lb. per sq. in. In Formulas 46 and 47,

f_r = compressive unit stress in metal core; and

R = least radius of gyration of the steel or cast-iron section.

The diameter of the cast-iron section shall not exceed one-half of the diameter of the core within the spiral. The spiral reinforcement shall be not less than 0.5% of the volume of the core within the spiral and shall conform in quality, spacing, and other requirements to the provisions for spirals in reinforced concrete columns. Ample sections of concrete and continuity of reinforcement shall be provided at the junction with beams or girders. The area of the concrete between the spiral and metal core shall be not less than that required to carry the total floor load of the story above on the basis of a stress in the concrete of $0.35f'_c$, unless special brackets are arranged on the metal core to receive directly the beam or slab loads.

171.—*Structural Steel Columns.*—The safe load on a structural steel column of a section which fully encloses or encases an area of concrete, and which is protected by an outside shell of concrete at least 3 in. thick, shall be computed in the same way as in the columns described in Section 170,

allowing $0.25f'_c$ on the area of the concrete enclosed by the steel section. The outside shell shall be reinforced by wire mesh or hoops weighing not less than 0.2 lb. per sq. ft. of surface of the core and with a maximum spacing of strands or hoops of 6 in. Special brackets shall be used to receive the entire floor load at each story. The working stress in steel columns shall be calculated by Formula 46, but shall not exceed 16 000 lb. per sq. in.

172.—*Long Columns.*—The permissible working unit stress on the core in axially loaded columns which have a length greater than 40 times the least radius of gyration of the column core ($40R$) shall be determined by the formula:

$$\frac{P'}{P} = 1.33 - \frac{h}{120\,R} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (48)$$

where P' = total safe axial load on long column;

P = total safe axial load on column of the same section whose $\frac{h}{R}$ is less than 40, determined as in Section 167; and

R = least radius of gyration of column core.

173.—*Bending Moments in Columns.*—The bending moments in interior and exterior columns shall be determined on the basis of loading conditions and end restraint and shall be provided for in the design.* The recognized standard methods shall be followed in calculating the stresses due to combined axial load and bending.

H.—Footings.

174.—*Types.*—Various types of reinforced concrete footings are in use, depending on conditions. The fundamental principles of the design of reinforced concrete will generally apply to footings as to other structural members. The requirements for flexure and shear in Sections 112 to 139, inclusive, shall govern the design of footings, except as hereinafter provided.

175—*Distribution of Pressure.*—The upward reaction per unit of area on the footing shall be taken as the column load divided by the area of base of the footing.

176.—*Pile Footing.*—Footings carried on piles shall be treated in the same manner as those bearing directly on the soil, except that the reaction shall be considered as a series of concentrated loads applied at the pile centers.

177.—*Sloped Footing.*—Footings in which the depth has been determined by the requirements for shear, as specified in Section 137, may be sloped between the critical section and the edge of the footing, provided that the shear on no section outside the critical section exceeds the value specified, and provided, further, that the thickness of the footing above the reinforcement at the edge shall not be less than 6 in. for footings on soil, nor less than 12 in. for footings on piles.

178.—*Stepped Footing.*—The top of the footing may be stepped instead of sloped, provided that the steps are so placed that the footing will have at

* The Committee is not prepared to make more definite recommendations at this time.

all sections a depth at least as great as that required for a sloping top. Stepped footings shall be cast monolithically.

179.—*Critical Section for Bending.*—In a concrete footing which supports a concrete column or pedestal, the critical section for bending shall be taken at the face of the column or pedestal. Where steel or cast-iron bases are used the moment in the footing shall be calculated at the edge of the base and at the center. In calculating this moment, the column or pedestal load shall be assumed as uniformly distributed over its base.

180.—*Square Column on Square Footing.*—For a square footing supporting a concentric square column, the bending moment at the critical section is that produced by the upward pressure on the trapezoid bounded by one face of the column, the corresponding outside edge of the footing, and the portions of the two diagonals. The center of application of the reaction on the two corner triangles of this trapezoid shall be taken at a distance from the face of the column equal to 0.6 of the projection of the footing. The center of the application of the reaction on the rectangular portion of the trapezoid shall be taken at its center of gravity. This gives a bending moment expressed by the formula:

$$M = \frac{w}{2}(a + 1.2c)\,c^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (49)$$

where M = bending moment at critical section of footing;
a = the width of face of column or pedestal;
c = projection of the footing from face of column; and
w = upward reaction per unit of area of base of footing.

(For typical footing designs, see Figs. 16 to 18*.)

181.—*Round Column on Square Footing.*—Square footings supporting a round or octagonal column shall be treated in the same manner as for a square column, using for the distance a the side of a square having an area equal to the area enclosed within the perimeter of the column.

182.—*Reinforcement.*—The reinforcement necessary to resist the bending moment in each direction in the footing shall be determined as for a reinforced concrete beam; the effective depth of the footing shall be the depth from the top to the plane of the reinforcement. The required area of reinforcement thus calculated shall be spaced uniformly across the footing, unless the footing width is greater than the side of the column or pedestal plus twice the effective depth of the footing, in which case the width over which the reinforcement is spread may be increased to include one-half the remaining width of the footing. In order that no considerable area of the footing shall remain unreinforced, additional bars shall be placed outside of the width specified, but such bars shall not be considered as effective in resisting the calculated bending moment. For the extra bars a spacing double that used for the reinforcement within the effective belt may be used.

183.—*Concrete Stress.*—The extreme fiber stress in compression in the concrete shall be kept within the limits specified in Section 198. The extreme fiber stress in sloped or stepped footings shall be based on the exact shape of

* Appendix II, p. 124.

the section for a width not greater than that assumed effective for reinforcement.

184.—*Irregular Footings.*—Rectangular or irregular-shaped footings shall be calculated by dividing the footings into rectangles or trapezoids tributary to the sides of the column, using the distance to the actual center of gravity of the area as the moment arm of the upward forces. Outstanding portions of combined footings shall be treated in the same manner. Other portions of combined footings shall be designed as beams or slabs.

185.—*Shearing Stresses.*—See Sections 137 to 139.

186.—*Bond Stresses.*—See Sections 142 to 144.

187.—*Transfer of Stress from Column Reinforcement.*—The compressive stress in longitudinal reinforcement in columns or pedestals shall be transferred to the footing by one of the following methods:

(*a*) By metal distributing bases having a sufficient area and thickness to transmit safely the load from the longitudinal reinforcement in compression and bending. The bases shall be accurately set and provided with a full bearing on the footing.

(*b*) By dowels, at least one for each bar and of total sectional area not less than the area of the longitudinal column reinforcement. The dowels shall project into the columns or into the pedestal or footing a distance not less than 50 times the diameter of the column bars.

188.—*Pedestals without Reinforcement.*—The allowable compressive unit stress on the gross area of a concentrically loaded pedestal without reinforcement shall not exceed $0.25f'_c$. If the column resting on such a pedestal is provided with distributing bases for the longitudinal reinforcement, the permissible compressive unit stress under the column core shall be determined by the following formula:

$$r_a = 0.25\, f'_c \sqrt[3]{\frac{A}{A'}} \qquad \text{(50)}$$

where r_a = permissible working stress over the loaded area;

A = total net area of the top of pedestal; and

A' = loaded area of pedestal.

189.—*Pedestals with Reinforcement.*—Where the permissible load at the top of a pedestal, determined by Formula 50, is less than the column load to be supported, dowels shall be used as specified in Section 187. If the height of the pedestal is not sufficient to give the required embedment to the dowels, they shall extend into the footing to a point 50 diameters below the top of the pedestal for plain bars and 40 diameters for deformed bars. If the column load divided by the cross-section of the pedestal exceeds $0.25f'_c$ the pedestal shall be considered as a section of a column and spiral reinforcement shall be provided accordingly.

190.—*Permissible Load at Top of Footings.*—Where distributing bases are used for transferring the stress from column reinforcement directly to the footing, the permissible compressive unit stress shall be determined by Formula 50. This formula may be applied by using A as the area of the top horizontal surface of the footing or with the following modifications:

(*a*) In footings, with sloping or stepped top in which a plane drawn from the edge of the base of the column so that it makes the greatest possible angle with the vertical, but remains entirely within the footing, has a slope with the horizontal not greater than 0.5, the total bearing area of the footing may be used for A.

(*b*) In footings in which the slope of the plane referred to is greater than 0.5, but not greater than 2.0, the permissible compressive unit stress at the top shall be determined by direct proportion, in terms of the slope, between the value found for a slope of 0.5 and the value of $0.25f'_c$ for a slope of 2.0. For a slope of 2.0 or greater the compressive unit stress at the top shall not exceed $0.25f'_c$.

(For typical footing designs, see Figs. 16 and 18*.)

191.—*Pedestal Footings.*—Pedestal footings may be designed as pedestals, that is, without reinforcement other than that required to transmit the column load, except that when supported directly on driven piles, a mat of reinforcing bars consisting of not less than 0.20 sq. in. per ft. of width in each direction shall be placed 3 in. above the top of the piles. The height of a pedestal footing shall not be greater than 4 times the average width.

I.—Reinforced Concrete Retaining Walls.

192.—*Types.*—Reinforced concrete retaining walls may be of the following types:

(*a*) Cantilever;
(*b*) Counterforted;
(*c*) Buttressed;
(*d*) Cellular.

193.—*Loads and Unit Stresses.*—Reinforced concrete retaining walls shall be designed† for the loads and reactions, and shall be so proportioned that the permissible unit stresses specified in Sections 196 to 208 are not exceeded. The heels of cantilever, counterforted, and buttressed retaining walls shall be proportioned for the maximum resultant vertical loads to which they will be subjected, but the sections shall be such that the normal permissible unit stresses will not be increased by more than 50% when the reaction from the foundation bed is neglected.

194.—*Details of Design.*—The following principles shall be followed in the design of reinforced concrete retaining walls:

(*a*) The unsupported toe and heel of the base slabs shall be considered as cantilever beams fixed at the edge of the support.

(*b*) The vertical section of a cantilever wall shall be considered as a cantilever beam fixed at the top of the base.

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* See Appendix II, p. 124.

† In proportioning retaining walls, consideration shall be given to the following:

(*a*) Maximum bearing pressure of soil;
(*b*) Uniformity of distribution of foundation pressure on yielding soils;
(*c*) Stability against sliding;
(*d*) Minor increase of the horizontal forces may seriously affect (*a*) and (*b*).

(c) The vertical sections of counterforted and buttressed walls and parts of base slabs supported by the counterforts or buttresses shall be designed in accordance with the requirements specified herein for the continuous slab.

(d) The exposed faces of walls without buttresses shall preferably be given a batter of not less than ¼ in. in 12 in.

(e) Counterforts shall be designed in accordance with the requirements specified for T-beams. Stirrups shall be provided in the counterforts to take the reaction from these spans when the tension reinforcement of the face walls and heels of bases is designed to span between the counterforts. Stirrups shall be anchored as near the exposed faces of the face walls, and as near the lower face of the bases, as practicable.

(f) Buttresses shall be designed in accordance with the requirements specified for rectangular beams.

(g) The shearing stress at the junction of the base with counterforts or buttresses shall not exceed the values specified in Sections 120 to 135.

(h) Horizontal metal reinforcement shall be well distributed of such form as to develop a high bond resistance. At least 0.25 sq. in. of horizontal metal reinforcement for each foot of height shall be provided near exposed surfaces not otherwise reinforced, to resist the formation of temperature and shrinkage cracks.

(i) Provision for temperature changes shall be made by grooved lock-joints spaced not over 60 ft. apart.

(j) Counterforts and buttresses, where used, shall be located under all points of concentrated loading, and at intermediate points spaced 8 to 12 ft. apart.

(k) The walls shall be cast monolithically between expansion joints, unless construction joints made in accordance with Sections 69 and 73 are provided.

195.—*Drains.*—Drains or "weep holes" not less than 4 in. in diameter and not more than 10 ft. apart, shall be provided. In counterforted walls there shall be at least one drain for each pocket formed by the counterforts.

J.—Floor-Slabs Supported on Four Sides.*

K.—Shrinkage and Temperature Stresses.*

L.—Summary of Working Stresses.

196.—*Notation.*—

f'_c = ultimate compressive strength of concrete at age of 28 days, based on tests of 6 by 12-in., or 8 by 16-in., cylinders, made and tested in accordance with the "Standard Methods of Making and Storing Specimens of Concrete in the Field" (Appendix XIV)† and the "Tentative Methods of Making Compression Tests of Concrete" (Appendix XIII).†

* The Committee is not now ready to report on these subjects.
† Not reproduced.

(a) Maximum Direct Stresses in Concrete.

197.—*Direct Compression.*—

(*a*) Columns whose length does not exceed $40R$:

1.—With spirals...varies with amount of longitudinal reinforcement
2.—Without spirals ....$0.20f'_c$

(*b*) 3.—Long columns...............................see Section 172

(*c*) Piers and pedestals:

1.—Without reinforcement ....$0.25f'_c$
2.—For special cases...............................see Section 188

198.—*Compression in Extreme Fiber.*—

(*a*) Extreme fiber stress in flexure..............................$0.40f'_c$
(*b*) Extreme fiber stress adjacent to supports of continuous beams..$0.45f'_c$

199.—*Bearing Compression.*—Anchorage of reinforcement..........$0.50f'_c$

200.—*Tension.*—All concrete members ....None

(b) Maximum Shearing Stresses in Concrete.

201.—*Beams without Web Reinforcement.*—

(*a*) Longitudinal bars anchored.................................. $0.03f'_c$
(*b*) Longitudinal bars not anchored.............................. $0.02f'_c$

202.—*Beams with Reinforcement.*—

(*a*) Beams with stirrups.........................see Sections 125 and 128
(*b*) Beams with bars bent up in several planes............see Section 125
(*c*) Beams with bars bent up in a single plane:

1.—Longitudinal bars anchored...................................$0.12f'_c$
2.—Longitudinal bars not anchored...............................$0.06f'_c$

203.—*Flat Slabs.*—

(*a*) Shear at distance, *d*, from capital or dropped panel.......... $0.03f'_c$
(*b*) Other limiting cases in flat slabs..................see Section 136

204.—*Footings.*—

(*a*) Longitudinal bars anchored.....................................$0.03f'_c$
(*b*) Longitudinal bars not anchored.............................. $0.02f'_c$

(c) Maximum Stresses in Reinforcement.

205.—*Tension in Steel.*—

(*a*) Billet-steel bars:

1.—Structural steel grade.................. 16 000 lb. per sq. in.
2.—Intermediate grade...................... 18 000 " " " "
3.—Hard grade.............................. 18 000 " " " "

(*b*) Rail-steel bars............................ 16 000 " " " "
(*c*) Structural steel........................... 16 000 " " " "
(*d*) Cold-drawn steel wire:

1.—Spirals ....stress not calculated.
2.—Elsewhere .... 18 000 lb. per sq. in.

206.—*Compression in Steel.*—

(*a*) Bars..............................same as Section 205 (*a*) and (*b*)

(*b*) Structural steel core of composite column......18 000 lb. per sq. in., reduced for slenderness ratio

(*c*) Structural steel column..........................16 000 lb. per sq. in., reduced for slenderness ratio

207.—*Compression in Cast Iron.*—

Composite columns with spirals.................... 10 000 lb. per sq. in.

(d) Maximum Bond Between Concrete and Steel.

208.—*Bond.*—

(*a*) Beams and slabs, plain bars.................................... $0.04f'_c$

(*b*) Beams and slabs, deformed bars............................... $0.05f'_c$

(*c*) Footings, plain bars, one-way................................... $0.04f'_c$

(*d*) Footings, deformed bars, one-way............................. $0.05f'_c$

(*e*) Footings, two-way reinforcement.. (*c*) or (*d*) reduced by 25 per cent.

(*f*) Footings, each additional direction of reinforcement.................. ....(*c*) or (*d*) reduced by 10 per cent.

TABLE 4.—PROPORTIONS* FOR CONCRETE OF GIVEN COMPRESSIVE STRENGTH AT 28 DAYS.

Table 4 gives the proportions in which Portland cement and a wide range in sizes of fine and coarse aggregates should be mixed to obtain concrete of compressive strengths ranging from 1 500 to 3 000 lb. per sq. in. at 28 days. Proportions are given for concrete of four different consistencies.

The purpose of the table is twofold:

1.—To furnish a guide in the selection of mixtures to be used in preliminary investigations of the strength of concrete from given materials.

2.—To indicate proportions which may be expected to produce concrete of a given strength under average conditions where control tests are not made.

If the proportions to be used in the work are selected from the table without preliminary tests of the materials, and control tests are not made during the progress of the work, the mixtures in bold-face type shall be used.

The use of this table as a guide in the selection of concrete mixtures is based on the following:

1.—Concrete shall be plastic;

2.—Aggregates shall be clean and structually sound;

3.—Aggregates shall be graded between the sizes indicated;

4.—Cement shall conform to the requirements of the Standard Specifications and Tests for Portland Cement (Serial Designation: C9—21) of the American Society for Testing Materials. (Appendix III.)†

The plasticity of the concrete shall be determined by the slump test carried out in accordance with the Tentative Specifications for Workability of Concrete for Concrete Pavements (Serial Designation: D62—20T) of the American Society for Testing Materials. (Appendix XII.)†

Apply the following rules in determining the size assigned to a given aggregate:

1. Not less than 15% shall be retained between the sieve which is considered the maximum size‡ and the next smaller sieve.

2.—Not more than 15% of a coarse aggregate shall be finer than the sieve considered as the minimum size.‡

3.—Only the sieve sizes given in the table shall be considered in applying rules (1) and (2).

4.—Sieve analysis shall be made in accordance with the Tentative Method of Test for Sieve Analysis of Aggregates for Concrete (Serial Designation: C41—21T) of the American Society for Testing Materials. (Appendix IX.)†

Proportions may be interpolated for concrete strengths, aggregate sizes and consistencies not covered by the table or determined by test.

* Based on the 28-day compressive strengths of 6 by 12-in. cylinders, made and stored in accordance with the Tentative Methods of Making Compression Tests of Concrete (Serial Designation: C39-21T) of the American Society for Testing Materials. (Appendix XIII.)

† Not reproduced.

‡ For example: a graded sand with 16% retained on the No. 8 sieve would fall in the 0-No. 4 size; if 14% or less were retained, the sand would fall in the 0-No. 8 size. A coarse aggregate having 16% coarser than 2-in. sieve would be considered as 3-in. aggregate.

TABLE 4 (*Continued*).—PROPORTIONS FOR 1 500 LB. PER SQ. IN. CONCRETE.

Proportions are expressed by volume as follows: Portland cement: fine aggregate: coarse aggregate.

Thus 1:2.6:4.6 indicates 1 part by volume of Portland cement, 2.6 parts by volume of fine aggregate and 4.6 parts by volume of coarse aggregate.

Size of coarse aggregate.	Slump, in inches	SIZE OF FINE AGGREGATE				
		0 – No. 28	0 – No. 14	0 – No. 8	0 – No. 4	0 – ⅜ in.
None.........	½ to 1	1 : 2.8	1 : 3.2	1 : 3.8	1 : 4.4	1 : 5.1
	3 " 4	1 : 2.4	1 : 2.8	1 : 3.3	1 : 3.8	1 : 4.5
	6 " 7	**1 : 1.9**	**1 : 2.2**	**1 : 2.6**	**1 : 3.0**	**1 : 3.6**
	8 "10	1 : 1.4	1 : 1.6	1 : 1.8	1 : 2.1	1 : 2.5
No. 4 to ¾ in...	½ to 1	1 : 2.6 : 4.6	1 : 2.9 : 4.3	1 : 3.4 : 4.1	1 : 3.9 : 3.6	1 : 4.6 : 3.1
	3 " 4	1 : 2.3 : 4.0	1 : 2.6 : 3.8	1 : 2.9 : 3.6	1 : 3.4 : 3.2	1 : 4.1 : 2.8
	6 " 7	**1 : 1.8 : 3.4**	**1 : 2.0 : 3.2**	**1 : 2.3 : 3.1**	**1 : 2.6 : 2.8**	**1 : 3.1 : 2.5**
	8 "10	1 : 1.1 : 2.5	1 : 1.3 : 2.4	1 : 1.5 : 2.4	1 : 1.7 : 2.2	1 : 2.1 : 2.0
No. 4 to 1 in....	½ to 1	1 : 2.4 : 5.3	1 : 2.7 : 5.2	1 : 3.1 : 5.0	1 : 3.5 : 4.7	1 : 4.3 : 4.3
	3 " 4	1 : 2.1 : 4.7	1 : 2.4 : 4.5	1 : 2.7 : 4.4	1 : 3.1 : 4.1	1 : 3.7 : 3.7
	6 " 7	**1 : 1.6 : 3.9**	**1 : 1.8 : 3.8**	**1 : 2.1 : 3.7**	**1 : 2.4 : 3.5**	**1 : 2.9 : 3.3**
	8 "10	1 : 1.1 : 2.9	1 : 1.2 : 2.8	1 : 1.4 : 2.8	1 : 1.6 : 2.7	1 : 1.9 : 2.5
No. 4 to 1½ in..	½ to 1	1 : 2.4 : 6.0	1 : 2.7 : 5.9	1 : 3.1 : 5.8	1 : 3.5 : 5.4	1 : 4.1 : 5.1
	3 " 4	1 : 2.0 : 5.4	1 : 2.3 : 5.3	1 : 2.7 : 5.2	1 : 3.0 : 5.0	1 : 3.5 : 4.6
	6 " 7	**1 : 1.6 : 4.4**	**1 : 1.8 : 4.3**	**1 : 2.0 : 4.3**	**1 : 2.3 : 4.1**	**1 : 2.7 : 3.9**
	8 "10	1 : 1.0 : 3.3	1 : 1.1 : 3.2	1 : 1.3 : 3.2	1 : 1.5 : 3.1	1 : 1.8 : 2.9
No. 4 to 2 in....	½ to 1	1 : 2.2 : 6.9	1 : 2.4 : 6.8	1 : 2.8 : 6.8	1 : 3.1 : 6.6	1 : 3.7 : 6.4
	3 " 4	1 : 1.8 : 6.2	1 : 2.0 : 6.1	1 : 2.4 : 6.1	1 : 2.7 : 6.0	1 : 3.1 : 5.7
	6 " 7	**1 : 1.4 : 5.1**	**1 : 1.6 : 5.0**	**1 : 1.8 : 5.0**	**1 : 2.0 : 5.0**	**1 : 2.4 : 4.8**
	8 "10	1 : 0.9 : 3.8	1 : 1.0 : 3.8	1 : 1.1 : 3.8	1 : 1.3 : 3.8	1 : 1.5 : 3.7
⅜ to 1 in......	½ to 1	1 : 2.8 : 5.2	1 : 3.1 : 5.1	1 : 3.6 : 4.8	1 : 4.2 : 4.6	1 : 4.8 : 4.1
	3 " 4	1 : 2.4 : 4.5	1 : 2.6 : 4.5	1 : 3.1 : 4.3	1 : 3.6 : 4.0	1 : 4.1 : 3.6
	6 " 7	**1 : 1.9 : 3.9**	**1 : 2.1 : 3.7**	**1 : 2.4 : 3.6**	**1 : 2.8 : 3.4**	**1 : 3.2 : 3.1**
	8 "10	1 : 1.3 : 2.8	1 : 1.4 : 2.8	1 : 1.6 : 2.7	1 : 1.9 : 2.6	1 : 2.2 : 2.4
⅜ to 1½ in....	½ to 1	1 : 2.8 : 5.8	1 : 3.1 : 5.7	1 : 3.5 : 5.5	1 : 4.1 : 5.3	1 : 4.7 : 4.9
	3 " 4	1 : 2.4 : 5.2	1 : 2.7 : 5.1	1 : 3.1 : 5.0	1 : 3.5 : 4.8	1 : 4.1 : 4.4
	6 " 7	**1 : 1.9 : 4.3**	**1 : 2.1 : 4.2**	**1 : 2.4 : 4.2**	**1 : 2.7 : 4.0**	**1 : 3.1 : 3.7**
	8 "10	1 : 1.2 : 3.2	1 : 1.4 : 3.2	1 : 1.6 : 3.1	1 : 1.8 : 3.0	1 : 2.1 : 2.9
⅜ to 2 in......	½ to 1	1 : 2.7 : 6.6	1 : 3.0 : 6.6	1 : 3.4 : 6.5	1 : 3.9 : 6.4	1 : 4.4 : 6.0
	3 " 4	1 : 2.3 : 5.9	1 : 2.6 : 5.9	1 : 2.9 : 5.8	1 : 3.3 : 5.6	1 : 3.7 : 5.5
	6 " 7	**1 : 1.8 : 4.9**	**1 : 2.0 : 4.8**	**1 : 2.2 : 4.8**	**1 : 2.6 : 4.8**	**1 : 3.0 : 4.5**
	8 "10	1 : 1.2 : 3.7	1 : 1.3 : 3.7	1 : 1.5 : 3.7	1 : 1.7 : 3.6	1 : 1.9 : 3.5
¾ to 1½ in....	½ to 1	1 : 3.2 : 5.4	1 : 3.6 : 5.3	1 : 4.1 : 5.1	1 : 4.7 : 4.8	1 : 5.3 : 4.4
	3 " 4	1 : 2.8 : 4.8	1 : 3.2 : 4.8	1 : 3.6 : 4.6	1 : 4.0 : 4.4	1 : 4.6 : 4.0
	6 " 7	**1 : 2.1 : 4.0**	**1 : 2.5 : 4.0**	**1 : 2.8 : 3.9**	**1 : 3.2 : 3.7**	**1 : 3.5 : 3.4**
	8 "10	1 : 1.5 : 3.0	1 : 1.7 : 3.0	1 : 1.9 : 2.9	1 : 2.2 : 2.8	1 : 2.5 : 2.7
¾ to 2 in.....	½ to 1	1 : 3.2 : 6.2	1 : 3.6 : 6.1	1 : 4.0 : 6.0	1 : 4.6 : 5.8	1 : 5.2 : 5.4
	3 " 4	1 : 2.8 : 5.5	1 : 3.1 : 5.5	1 : 3.5 : 5.4	1 : 3.9 : 5.2	1 : 4.5 : 4.9
	6 " 7	**1 : 2.1 : 4.5**	**1 : 2.4 : 4.6**	**1 : 2.7 : 4.5**	**1 : 3.1 : 4.4**	**1 : 3.5 : 4.1**
	8 "10	1 : 1.4 : 3.4	1 : 1.6 : 3.4	1 : 1.8 : 3.4	1 : 2.1 : 3.4	1 : 2.4 : 3.3
¾ to 3 in......	½ to 1	1 : 3.2 : 7.1	1 : 3.6 : 7.1	1 : 4.0 : 7.0	1 : 4.6 : 6.9	1 : 5.2 : 6.6
	3 " 4	1 : 2.7 : 6.3	1 : 3.0 : 6.3	1 : 3.4 : 6.3	1 : 4.0 : 6.2	1 : 4.5 : 5.9
	6 " 7	**1 : 2.1 : 5.1**	**1 : 2.4 : 5.2**	**1 : 2.7 : 5.2**	**1 : 3.1 : 6.1**	**1 : 3.5 : 4.9**
	8 "10	1 : 1.4 : 3.8	1 : 1.6 : 3.9	1 : 1.8 : 3.9	1 : 2.1 : 3.9	1 : 2.4 : 3.8

TABLE 4 (*Continued*).—PROPORTIONS FOR 2 000 LB. PER SQ. IN. CONCRETE.

Proportions are expressed by volume as follows: Portland cement: fine aggregate: coarse aggregate.

Thus 1:2.6:4.6 indicates 1 part by volume of Portland cement, 2.6 parts by volume of fine aggregate and 4.6 parts by volume of coarse aggregate.

Size of coarse aggregate	Slump, in inches	Size of Fine Aggregate				
		0 – No. 28	0 – No. 14	0 – No. 8	0 – No. 4	0 – ⅜ in.
None	½ to 1	1 : 2.2	1 : 2.6	1 : 3.0	1 : 3.5	1 : 4.1
	3 " 4	1 : 1.9	1 : 2.2	1 : 2.6	1 : 3.0	1 : 3.5
	6 " 7	1 : 1.5	1 : 1.7	1 : 2.0	1 : 2.3	1 : 2.7
	8 "10	1 : 1.0	1 : 1.1	1 : 1.3	1 : 1.6	1 : 1.8
No. 4 to ¾ in.	½ to 1	1 : 2.1 : 3.8	1 : 2.3 : 3.7	1 : 2.6 : 3.5	1 : 3.0 : 3.1	1 : 3.6 : 2.8
	3 " 4	1 : 1.7 : 3.3	1 : 1.9 : 3.2	1 : 2.2 : 3.1	1 : 2.6 : 2.8	1 : 3.0 : 2.4
	6 " 7	1 : 1.3 : 2.7	1 : 1.4 : 2.6	1 : 1.7 : 2.5	1 : 1.9 : 2.3	1 : 2.3 : 2.1
	8 "10	1 : 0.8 : 1.9	1 : 0.9 : 1.9	1 : 1.0 : 1.8	1 : 1.2 : 1.7	1 : 1.5 : 1.6
No. 4 to 1 in.	½ to 1	1 : 1.9 : 4.5	1 : 2.2 : 4.3	1 : 2.5 : 4.2	1 : 2.8 : 3.9	1 : 3.4 : 3.6
	3 " 4	1 : 1.6 : 3.9	1 : 1.8 : 3.8	1 : 2.1 : 3.7	1 : 2.4 : 3.5	1 : 2.8 : 3.2
	6 " 7	1 : 1.2 : 3.1	1 : 1.3 : 3.1	1 : 1.5 : 3.0	1 : 1.8 : 2.9	1 : 2.1 : 2.7
	8 "10	1 : 0.7 : 2.2	1 : 0.8 : 2.2	1 : 1.0 : 2.3	1 : 1.1 : 2.1	1 : 1.3 : 2.0
No. 4 to 1½ in.	½ to 1	1 : 1.9 : 5.0	1 : 2.1 : 4.9	1 : 2.4 : 4.9	1 : 2.7 : 4.6	1 : 3.2 : 4.4
	3 " 4	1 : 1.6 : 4.4	1 : 1.7 : 4.3	1 : 2.0 : 4.2	1 : 2.4 : 4.0	1 : 2.7 : 3.8
	6 " 7	1 : 1.1 : 3.5	1 : 1.3 : 3.5	1 : 1.4 : 3.5	1 : 1.7 : 3.4	1 : 2.0 : 3.2
	8 "10	1 : 0.7 : 2.5	1 : 0.8 : 2.5	1 : 0.9 : 2.5	1 : 1.0 : 2.4	1 : 1.2 : 2.3
No. 4 to 2 in.	½ to 1	1 : 1.7 : 5.8	1 : 1.9 : 5.7	1 : 2.1 : 5.8	1 : 2.4 : 5.6	1 : 2.8 : 5.5
	3 " 4	1 : 1.4 : 5.0	1 : 1.5 : 5.0	1 : 1.8 : 5.0	1 : 2.0 : 4.9	1 : 2.3 : 4.7
	6 " 7	1 : 1.0 : 4.1	1 : 1.1 : 4.1	1 : 1.2 : 4.1	1 : 1.4 : 4.1	1 : 1.7 : 3.9
	8 "10	1 : 0.6 : 2.9	1 : 0.7 : 2.9	1 : 0.7 : 3.0	1 : 0.8 : 2.9	1 : 1.0 : 2.9
⅜ to 1 in.	½ to 1	1 : 2.2 : 4.4	1 : 2.5 : 4.2	1 : 2.8 : 4.1	1 : 3.3 : 3.8	1 : 3.8 : 3.4
	3 " 4	1 : 1.9 : 3.8	1 : 2.1 : 3.7	1 : 2.4 : 3.6	1 : 2.8 : 3.4	1 : 3.2 : 3.1
	6 " 7	1 : 1.4 : 3.1	1 : 1.5 : 3.0	1 : 1.8 : 3.0	1 : 2.1 : 2.8	1 : 2.4 : 2.5
	8 "10	1 : 0.9 : 2.2	1 : 1.0 : 2.2	1 : 1.1 : 2.2	1 : 1.3 : 2.0	1 : 1.5 : 1.9
⅜ to 1½ in.	½ to 1	1 : 2.2 : 4.9	1 : 2.5 : 4.8	1 : 2.8 : 4.7	1 : 3.2 : 4.6	1 : 3.7 : 4.2
	3 " 4	1 : 1.9 : 4.3	1 : 2.1 : 4.2	1 : 2.4 : 4.1	1 : 2.7 : 4.0	1 : 3.1 : 3.7
	6 " 7	1 : 1.4 : 3.5	1 : 1.5 : 3.4	1 : 1.7 : 3.4	1 : 2.0 : 3.3	1 : 2.3 : 3.1
	8 "10	1 : 0.9 : 2.5	1 : 1.0 : 2.5	1 : 1.1 : 2.4	1 : 1.3 : 2.4	1 : 1.5 : 2.3
⅜ to 2 in.	½ to 1	1 : 2.1 : 5.6	1 : 2.3 : 5.5	1 : 2.6 : 5.5	1 : 3.0 : 5.4	1 : 3.5 : 5.1
	3 " 4	1 : 1.7 : 4.8	1 : 2.0 : 4.8	1 : 2.2 : 4.8	1 : 2.5 : 4.7	1 : 2.9 : 4.4
	6 " 7	1 : 1.3 : 4.0	1 : 1.4 : 3.9	1 : 1.6 : 3.9	1 : 1.8 : 3.9	1 : 2.1 : 3.8
	8 "10	1 : 0.8 : 2.9	1 : 0.9 : 2.9	1 : 1.0 : 2.9	1 : 1.2 : 2.9	1 : 1.3 : 2.8
¾ to 1½ in.	½ to 1	1 : 2.6 : 4.5	1 : 2.9 : 4.5	1 : 3.3 : 4.4	1 : 3.8 : 4.2	1 : 4.3 : 3.9
	3 " 4	1 : 2.2 : 3.9	1 : 2.5 : 3.9	1 : 2.8 : 3.8	1 : 3.2 : 3.6	1 : 3.6 : 3.3
	6 " 7	1 : 1.6 : 3.2	1 : 1.8 : 3.2	1 : 2.1 : 3.1	1 : 2.4 : 3.0	1 : 2.7 : 2.8
	8 "10	1 : 1.0 : 2.3	1 : 1.2 : 2.3	1 : 1.4 : 2.2	1 : 1.6 : 2.2	1 : 1.8 : 2.1
¾ to 2 in.	½ to 1	1 : 2.5 : 5.2	1 : 2.8 : 5.2	1 : 3.2 : 5.1	1 : 3.6 : 5.0	1 : 4.1 : 4.7
	3 " 4	1 : 2.1 : 4.5	1 : 2.4 : 4.5	1 : 2.7 : 4.4	1 : 3.1 : 4.3	1 : 3.5 : 4.0
	6 " 7	1 : 1.6 : 3.7	1 : 1.8 : 3.7	1 : 2.0 : 3.7	1 : 2.3 : 3.6	1 : 2.6 : 3.5
	8 "10	1 : 1.0 : 2.6	1 : 1.1 : 2.7	1 : 1.3 : 2.6	1 : 1.5 : 2.7	1 : 1.7 : 2.6
¾ to 3 in.	½ to 1	1 : 2.5 : 6.0	1 : 2.9 : 5.9	1 : 3.2 : 5.9	1 : 3.6 : 5.8	1 : 4.1 : 5.6
	3 " 4	1 : 2.1 : 5.1	1 : 2.4 : 5.2	1 : 2.7 : 5.2	1 : 3.1 : 5.1	1 : 3.5 : 4.9
	6 " 7	1 : 1.5 : 4.1	1 : 1.7 : 4.2	1 : 2.0 : 4.2	1 : 2.3 : 4.2	1 : 2.5 : 4.0
	8 "10	1 : 1.0 : 2.9	1 : 1.1 : 3.0	1 : 1.3 : 3.0	1 : 1.5 : 3.0	1 : 1.7 : 3.0

TABLE 4 (*Continued*).—PROPORTIONS FOR 2 500 LB. PER SQ. IN. CONCRETE.

Proportions are expressed by volume as follows: Portland cement: fine aggregate: coarse aggregate.

Thus 1:2.6:4.6 indicates 1 part by volume of Portland cement, 2.6 parts by volume of fine aggregate and 4.6 parts by volume of coarse aggregate.

Size of coarse aggregate	Slump, in inches	SIZE OF FINE AGGREGATE				
		0 – No. 28	0 – No. 14	0 – No. 8	0 – No. 4	0 – ⅜ in.
None..........	½ to 1	1 : 1.8	1 : 2.1	1 : 2.4	1 : 2.9	1 : 3.3
	3 " 4	1 : 1.5	1 : 1.8	1 : 2.1	1 : 2.4	1 : 2.8
	6 " 7	**1 : 1.1**	**1 : 1.3**	**1 : 1.6**	**1 : 1.8**	**1 : 2.1**
	8 "10	1 : 0.7	1 : 0.8	1 : 0.9	1 : 1.1	1 : 1.3
No. 4 to ¾ in...	½ to 1	1 : 1.6 : 3.2	1 : 1.8 : 3.1	1 : 2.1 : 3.0	1 : 2.4 : 2.7	1 : 2.9 : 2.4
	3 " 4	1 : 1.3 : 2.8	1 : 1.5 : 2.7	1 : 1.7 : 2.6	1 : 2.0 : 2.4	1 : 2.4 : 2.2
	6 " 7	**1 : 1.0 : 2.2**	**1 : 1.1 : 2.2**	**1 : 1.3 : 2.1**	**1 : 1.5 : 2.0**	**1 : 1.8 : 1.8**
	8 "10	1 : 0.5 : 1.4	1 : 0.6 : 1.4	1 : 0.7 : 1.4	1 : 0.8 : 1.4	1 : 1.0 : 1.3
No. 4 to 1 in....	½ to 1	1 : 1.5 : 3.7	1 : 1.7 : 3.7	1 : 2.0 : 3.5	1 : 2.2 : 3.4	1 : 2.7 : 3.1
	3 " 4	1 : 1.2 : 3.3	1 : 1.4 : 3.2	1 : 1.6 : 3.1	1 : 1.9 : 3.0	1 : 2.2 : 2.7
	6 " 7	**1 : 0.9 : 2.6**	**1 : 1.0 : 2.5**	**1 : 1.1 : 2.5**	**1 : 1.3 : 2.4**	**1 : 1.6 : 2.3**
	8 "10	1 : 0.5 : 1.7	1 : 0.6 : 1.7	1 : 0.6 : 1.7	1 : 0.7 : 1.6	1 : 0.9 : 1.5
No. 4 to 1½ in..	½ to 1	1 : 1.4 : 4.2	1 : 1.6 : 4.1	1 : 1.9 : 4.1	1 : 2.2 : 4.0	1 : 2.5 : 3.8
	3 " 4	1 : 1.2 : 3.7	1 : 1.3 : 3.6	1 : 1.5 : 3.6	1 : 1.8 : 3.5	1 : 2.1 : 3.3
	6 " 7	**1 : 0.9 : 2.9**	**1 : 0.9 : 2.8**	**1 : 1.1 : 2.8**	**1 : 1.3 : 2.8**	**1 : 1.5 : 2.6**
	8 "10	1 : 0.5 : 1.9	1 : 0.5 : 1.9	1 : 0.6 : 1.9	1 : 0.7 : 1.8	1 : 0.8 : 1.8
No. 4 to 2 in...	½ to 1	1 : 1.3 : 4.9	1 : 1.4 : 4.8	1 : 1.6 : 4.9	1 : 1.9 : 4.8	1 : 2.2 : 4.7
	3 " 4	1 : 1.1 : 4.3	1 : 1.2 : 4.2	1 : 1.3 : 4.3	1 : 1.6 : 4.2	1 : 1.8 : 4.1
	6 " 7	**1 : 0.7 : 3.3**	**1 : 0.8 : 3.3**	**1 : 0.9 : 3.4**	**1 : 1.1 : 3.3**	**1 : 1.2 : 3.3**
	8 "10	1 : 0.4 : 2.2	1 : 2.4 : 2.2	1 : 0.5 : 2.2	1 : 0.6 : 2.2	1 : 0.6 : 2.2
⅜ to 1 in......	½ to 1	1 : 1.8 : 3.7	1 : 2.0 : 3.6	1 : 2.3 : 3.5	1 : 2.6 : 3.3	1 : 3.0 : 2.9
	3 " 4	1 : 1.4 : 3.2	1 : 1.6 : 3.1	1 : 1.9 : 2.9	1 : 2.2 : 2.9	1 : 2.5 : 2.6
	6 " 7	**1 : 1.0 : 2.5**	**1 : 1.2 : 2.5**	**1 : 1.3 : 2.4**	**1 : 1.6 : 2.3**	**1 : 1.8 : 2.2**
	8 "10	1 : 0.6 : 1.6	1 : 0.7 : 1.6	1 : 0.8 : 1.6	1 : 0.9 : 1.6	1 : 1.0 : 1.5
⅜ to 1½ in....	½ to 1	1 : 1.7 : 4.1	1 : 1.9 : 4.1	1 : 2.2 : 4.0	1 : 2.5 : 3.9	1 : 2.9 : 3.6
	3 " 4	1 : 1.5 : 3.6	1 : 1.6 : 3.6	1 : 1.8 : 3.5	1 : 2.1 : 3.4	1 : 2 3 : 3.2
	6 " 7	**1 : 1.0 : 2.9**	**1 : 1.2 : 2.8**	**1 : 1.3 : 2.8**	**1 : 1.5 : 2.7**	**1 : 1.8 : 2.6**
	8 "10	1 : 0.6 : 1.9	1 : 0.6 : 1.9	1 : 0.8 : 1.8	1 : 0.9 : 1.8	1 : 1.0 : 1.8
⅜ to 2 in......	½ to 1	1 : 1.7 : 4.7	1 : 1.8 : 4.7	1 : 2.1 : 4.7	1 : 2.4 : 4.6	1 : 2.7 : 4.4
	3 " 4	1 : 1.4 : 4.1	1 : 1.5 : 4.1	1 : 1.7 : 4.1	1 : 2.0 : 4.0	1 : 2.3 : 3.9
	6 " 7	**1 : 1.0 : 3.2**	**1 : 1.1 : 3.2**	**1 : 1.2 : 3.2**	**1 : 1.4 : 3.2**	**1 : 1.6 : 3.1**
	8 "10	1 : 0.5 : 2.1	1 : 0.6 : 2.1	1 : 0.7 : 2.2	1 : 0.8 : 2.2	1 : 0.9 : 2.1
¾ to 1½ in....	½ to 1	1 : 2.0 : 3.8	1 : 2.3 : 3.8	1 : 2.6 : 3.7	1 : 3.0 : 3.6	1 : 3.4 : 3.3
	3 " 4	1 : 1.7 : 3.3	1 : 2.0 : 3.3	1 : 2.2 : 3.2	1 : 2.5 : 3.2	1 : 2.9 : 2.9
	6 " 7	**1 : 1.2 : 2.6**	**1 : 1.4 : 2.6**	**1 : 1.6 : 2.6**	**1 : 1.9 : 2.5**	**1 : 2.1 : 2.3**
	8 "10	1 : 0.7 : 1.7	1 : 0.8 : 1.7	1 : 0.9 : 1.7	1 : 1.1 : 1.7	1 : 1.2 : 1.6
¾ to 2 in......	½ to 1	1 : 2.0 : 4.4	1 : 2.2 : 4.4	1 : 2.5 : 4.3	1 : 2.9 : 4.3	1 : 3.3 : 4.1
	3 " 4	1 : 1.7 : 3.8	1 : 1.9 : 3.8	1 : 2.1 : 3.8	1 : 2.5 : 3.7	1 : 2.8 : 3.6
	6 " 7	**1 : 1.2 : 3.0**	**1 : 1.4 : 3.0**	**1 : 1.5 : 3.0**	**1 : 1.8 : 3.0**	**1 : 2.0 : 2.8**
	8 "10	1 : 0.7 : 2.0	1 : 0.8 : 2.0	1 : 0.9 : 2.0	1 : 1.0 : 2.0	1 : 1.2 : 2.0
¾ to 3 in......	½ to 1	1 : 2.0 : 5.0	1 : 2.2 : 5.0	1 : 2.5 : 5.0	1 : 2.7 : 5.0	1 : 3.2 : 4.7
	3 " 4	1 : 1.7 : 4.3	1 : 1.9 : 4.3	1 : 2.1 : 4.3	1 : 2.4 : 4.3	1 : 2.7 : 4.1
	6 " 7	**1 : 1.2 : 3.3**	**1 : 1.4 : 3.4**	**1 : 1.5 : 3.4**	**1 : 1.8 : 3.4**	**1 : 2.0 : 3.3**
	8 "10	1 : 0.7 : 2.2	1 : 0 8 : 2.2	1 : 0.9 : 2.2	1 : 1.0 : 2.3	1 : 1.2 : 2.3

TABLE 4 (*Continued*).—PROPORTIONS FOR 3 000 LB. PER SQ. IN. CONCRETE.

Proportions are expressed by volume as follows: Portland cement: fine aggregate: coarse aggregate.

Thus 1:2.6:4.6 indicates 1 part by volume of Portland cement, 2.6 parts by volume of fine aggregate and 4.6 parts by volume of coarse aggregate.

Size of coarse aggregate	Slump, in inches	SIZE OF FINE AGGREGATE				
		0 – No. 28	0 – No. 14	0 – No. 8	0 – No. 4	0 – ⅜ in.
None..........	½ to 1	1 : 1.5	1 : 1.7	1 : 2.0	1 : 2.3	1 : 2.7
	3 " 4	1 : 1.2	1 : 1.4	1 : 1.7	1 : 1.9	1 : 2.3
	6 " 7	1 : 0.9	1 : 1.0	1 : 1.2	1 : 1.4	1 : 1.6
	8 "10	1 : 0.5	1 : 0.6	1 : 0.7	1 : 0.8	1 : 0.9
No. 4 to ¾ in...	½ to 1	1 : 1.3 : 2.7	1 : 1.5 : 2.6	1 : 1.7 : 2.5	1 : 1.9 : 2.4	1 : 2.3 : 2.1
	3 " 4	1 : 1.0 : 2.3	1 : 1.2 : 2.2	1 : 1.4 : 2.2	1 : 1.6 : 2.0	1 : 1.9 : 1.8
	6 " 7	1 : 0.7 : 1.7	1 : 0.8 : 1.7	1 : 0.9 : 1.7	1 : 1.1 : 1.6	1 : 1.3 : 1.4
	8 "10	1 : 0.3 : 1.0	1 : 0.4 : 1.0	1 : 0.5 : 1.0	1 : 0.5 : 1.0	1 : 0.6 : 0.9
No. 4 to 1 in....	½ to 1	1 : 1.2 : 3.1	1 : 1.3 : 3.1	1 : 1.5 : 3.0	1 : 1.8 : 2.9	1 : 2.1 : 2.7
	3 " 4	1 : 0.9 : 2.7	1 : 1.1 : 2.6	1 : 1.2 : 2.6	1 : 1.4 : 2.5	1 : 1.7 : 2.3
	6 " 7	1 : 0.6 : 2.0	1 : 0.7 : 2.0	1 : 0.8 : 2.0	1 : 0.9 : 1.9	1 : 1.1 : 1.8
	8 "10	1 : 0.3 : 1.2	1 : 0.3 : 1.2	1 : 0.4 : 1.2	1 : 0.5 : 1.2	1 : 0.6 : 1.2
No. 4 to 1½ in..	½ to 1	1 : 1.1 : 3.6	1 : 1.2 : 3.5	1 : 1.5 : 3.5	1 : 1.7 : 3.4	1 : 2.0 : 3.2
	3 " 4	1 : 0.9 : 3.0	1 : 1.0 : 2.9	1 : 1.2 : 2.9	1 : 1.4 : 2.9	1 : 1.6 : 2.7
	6 " 7	1 : 0.6 : 2.2	1 : 0.7 : 2.2	1 : 0.8 : 2.2	1 : 0.9 : 2.2	1 : 1.1 : 2.1
	8 "10	1 : 0.3 : 1.4	1 : 0.3 : 1.3	1 : 0.4 : 1.4	1 : 0.5 : 1.4	1 : 0.5 : 1.3
No. 4 to 2 in...	½ to 1	1 : 1.0 : 4.1	1 : 1.1 : 4.1	1 : 1.2 : 4.1	1 : 1.4 : 4.1	1 : 1.6 : 4.0
	3 " 4	1 : 0.8 : 3.4	1 : 0.9 : 3.4	1 : 1.0 : 3.5	1 : 1.1 : 3.4	1 : 1.3 : 3.4
	6 " 7	1 : 0.5 : 2.6	1 : 0.6 : 2.6	1 : 0.6 : 2.7	1 : 0.7 : 2.6	1 : 0.9 : 2.6
	8 "10	1 : 0.2 : 1.6	1 : 0.3 : 1.6	1 : 0.3 : 1.7	1 : 0.4 : 1.7	1 : 0.4 : 1.7
⅜ to 1 in......	½ to 1	1 : 1.4 : 3.1	1 : 1.5 : 3.0	1 : 1.8 : 2.9	1 : 2.1 : 2.8	1 : 2.4 : 2.6
	3 " 4	1 : 1.1 : 2.6	1 : 1.3 : 2.6	1 : 1.5 : 2.5	1 : 1.7 : 2.4	1 : 2.0 : 2.2
	6 " 7	1 : 0.8 : 2.0	1 : 0.8 : 2.0	1 : 1.0 : 1.9	1 : 1.1 : 1.9	1 : 1.3 : 1.8
	8 "10	1 : 0.4 : 1.2	1 : 0.4 : 1.2	1 : 0.5 : 1.2	1 : 0.6 : 1.2	1 : 0.7 : 1.1
⅜ to 1½ in....	½ to 1	1 : 1.4 : 3.5	1 : 1.5 : 3.4	1 : 1.7 : 3.4	1 : 2.0 : 3.3	1 : 2.3 : 3.1
	3 " 4	1 : 1.1 : 3.0	1 : 1.2 : 2.9	1 : 1.4 : 2.9	1 : 1.6 : 2.8	1 : 1.9 : 2.6
	6 " 7	1 : 0.6 : 2.2	1 : 0.8 : 2.2	1 : 1.0 : 2.2	1 : 1.1 : 2.1	1 : 1.3 : 2.0
	8 "10	1 : 0.4 : 1.4	1 : 0.4 : 1.4	1 : 0.5 : 1.4	1 : 0.6 : 1.3	1 : 0.7 : 1.3
⅜ to 2 in......	½ to 1	1 : 1.3 : 4.0	1 : 1.4 : 4.0	1 : 1.6 : 4.0	1 : 1.9 : 3.9	1 : 2.1 : 3.8
	3 " 4	1 : 1.0 : 3.4	1 : 1.2 : 3.4	1 : 1.3 : 3.3	1 : 1.5 : 3.3	1 : 1.7 : 3.2
	6 " 7	1 : 0.7 : 2.6	1 : 0.8 : 2.5	1 : 0.9 : 2.6	1 : 1.0 : 2.6	1 : 1.1 : 2.5
	8 "10	1 : 0.4 : 1.6	1 : 0.4 : 1.6	1 : 0.5 : 1.6	1 : 0.5 : 1.6	1 : 0.6 : 1.6
¾ to 1½ in....	½ to 1	1 : 1.6 : 3.2	1 : 1.8 : 3.2	1 : 2.1 : 3.2	1 : 2.4 : 3.1	1 : 2.7 : 2.9
	3 " 4	1 : 1.3 : 2.7	1 : 1.5 : 2.7	1 : 1.7 : 2.7	1 : 2.0 : 2.6	1 : 2.3 : 2.5
	6 " 7	1 : 0.9 : 2.0	1 : 1.0 : 2.1	1 : 1.2 : 2.0	1 : 1.4 : 2.0	1 : 1.5 : 1.8
	8 "10	1 : 0.5 : 1.2	1 : 0.5 : 1.3	1 : 0.6 : 1.3	1 : 0.7 : 1.3	1 : 0.8 : 1.2
¾ to 2 in......	½ to 1	1 : 1.6 : 3.7	1 : 1.8 : 3.7	1 : 2.0 : 3.7	1 : 2.4 : 3.6	1 : 2.6 : 3.5
	3 " 4	1 : 1.3 : 3.1	1 : 1.5 : 3.1	1 : 1.6 : 3.1	1 : 1.9 : 3.1	1 : 2.2 : 3.0
	6 " 7	1 : 0.9 : 2.4	1 : 1.1 : 2.4	1 : 1.1 : 2.4	1 : 1.3 : 2.4	1 : 1.5 : 2.3
	8 "10	1 : 0.5 : 1.5	1 : 0.5 : 1.5	1 : 0.6 : 1.5	1 : 0.7 : 1.5	1 : 0.8 : 1.5
¾ to 3 in......	½ to 1	1 : 1.6 : 4.2	1 : 1.8 : 4.2	1 : 2.0 : 4.2	1 : 2.3 : 4.1	1 : 2.6 : 4.0
	3 " 4	1 : 1.3 : 3.5	1 : 1.5 : 3.6	1 : 1.6 : 3.6	1 : 1.9 : 3.6	1 : 2.1 : 3.5
	6 " 7	1 : 0.9 : 2.6	1 : 1.0 : 2.6	1 : 1.1 : 2.6	1 : 1.3 : 2.6	1 : 1.4 : 2.6
	8 "10	1 : 0.5 : 1.6	1 : 0.5 : 1.6	1 : 0.6 : 1.7	1 : 0.7 : 1.7	1 : 0.8 : 1.7

APPENDIX I.

STANDARD NOTATION.

All symbols used in the Tentative Specifications for Concrete and Reinforced Concrete have been collected here for convenience of reference. The symbols are in general defined in the text near the point where formulas occur. In a few instances the same symbol is used in two distinct senses; however, there is little danger of confusion from this source.

a = spacing of web reinforcement bars measured perpendicular to their direction (see Section 135);

a = width of face of column or pedestal;

α = angle between inclined web bars and longitudinal bars;

A = total net area of column, footing, or pedestal, exclusive of fire-proofing;

A' = loaded area of pedestal, pier or footing;

A_c = $A\ (1-p)$ = net area of concrete core of column;

A'_c = net area of concrete in columns (total column area minus steel area);

A_s = effective cross-sectional area of metal reinforcement in tension in beams or compression in columns; and the effective cross-sectional area of metal reinforcement which crosses any of the principal design sections of a flat slab and which meets the requirements of Section 156;

A_v = total area of web reinforcement in tension within a distance of a (a_1, a_2, a_3, etc.) or the total area of all bars bent up in any one plane;

b = width of rectangular beam or width of flange of T-beam;

b' = width of stem of T-beam;

b_1 = dimension of the dropped panel of a flat slab in the direction parallel to l_1;*

c = base diameter of the largest right circular cone which lies entirely within the column (including the capital) whose vertex angle is 90° and whose base is 1½ in. below the bottom of the slab or the bottom of the dropped panel (see Fig. 14†);

c = projection of footing from face of column;

C = total compressive stress in concrete;

C' = total compressive stress in reinforcement;

d = depth from compression surface of beam or slab to center of longitudinal tension reinforcement;

d' = depth from compression surface of beam or slab to center of compression reinforcement;

* In flat slab design, the column strip and the middle strip to be used when considering moments in the direction of the dimension l are located and dimensioned as shown in Fig. 15. The dimension l_1 does not always represent the short length of the panel. When moments in the direction of the shorter panel length are considered, the dimensions l and l_1 are to be interchanged and strips corresponding to those shown in Fig. 15 but extending in the direction of the shorter panel length are to be considered.

† Appendix II, p. 123.

E_c = modulus of elasticity of concrete in compression;
E_s = modulus of elasticity of steel in tension = 30 000 000 lb. per sq. in.
f_c = compressive unit stress in extreme fiber of concrete;
f'_c = ultimate compressive strength of concrete at age of 28 days, based on tests of 6 by 12-in. or 8 by 16-in. cylinders made and tested in accordance with the Standard Methods of Making and Storing Specimens of Concrete in the Field (Appendix XIV*) and the Tentative Methods of Making Compression Tests of Concrete (Appendix XIII*);
f_r = compressive unit stress in metal core;
f_s = tensile unit stress in longitudinal reinforcement;
f'_s = compressive unit stress in longitudinal reinforcement;
f_v = tensile unit stress in web reinforcement;
h = unsupported length of column;
I = moment of inertia of a section about the neutral axis for bending;
j = ratio of lever arm of resisting couple to depth, d;
jd = $d-z$ = arm of resisting couple;
k = ratio of depth of neutral axis to depth, d;
l = span length of beam or slab (general distance from center to center of supports; for special cases, see Sections 108 and 148);
l = span length of flat slab, center to center of columns, in the rectangular direction in which moments are considered;†
l_1 = span length of flat slab, center to center of columns, perpendicular to the rectangular direction in which moments are considered;†
M = bending moment or moment of resistance in general;
M_0 = sum of positive and negative bending moments in either rectangular direction, at the principal design sections of a panel of a flat slab;
n = $\frac{E_s}{E_c}$ = ratio of modulus of elasticity of steel to that of concrete;
o = perimeter of bar;
Σo = sum of perimeters of bars in one set;
p = ratio of effective area of tension reinforcement to effective area of concrete in beams = $\frac{A_s}{bd}$; and the ratio of effective area of longitudinal reinforcement to the area of the concrete core in columns;
p' = ratio of effective area of compression reinforcement to effective area of concrete in beams;
P = total safe axial load on columns whose $\frac{h}{R}$ is less than 40;
P' = total safe axial load on long column;
r = ratio of cross-sectional area of negative reinforcement which crosses entirely over the column capital of a flat slab or over the dropped panel, to the total cross-sectional area of the negative reinforcement in the two-column strips;

* Not reproduced.
† See footnote, p. 116.

r_a = permissible working stress in concrete over the loaded area of a pedestal, pier or footing;

R = ratio of positive or negative moment in two-column strips or one middle strip of a flat slab, to M_0;

R = least radius of gyration of a section;

s = spacing of web members, measured at the neutral axis and in the direction of the longitudinal axis of the beam;

t = thickness of flange of T-beam;

t_1 = thickness of flat slab without dropped panels or thickness of a dropped panel (see Fig. 14*);

t_2 = thickness of flat slab with dropped panels at points away from the dropped panel (see Fig. 14*);

T = total tensile stress in longitudinal reinforcement;

u = bond stress per unit of area of surface of bar;

v = shearing unit stress;

V = total shear;

V' = external shear on any section after deducting that carried by the concrete;

w = uniformly distributed load per unit of length of beam or slab;

w = upward reaction per unit of area of base of footing;

w' = uniformly distributed dead and live load per unit of area of a floor or roof;

W = total dead and live load uniformly distributed over a single panel area;

z = depth from compression surface of beam or slab to resultant of compressive stresses.

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* Appendix II, p. 123.

APPENDIX II.

See Appendix I for explanation of symbols used in figures.

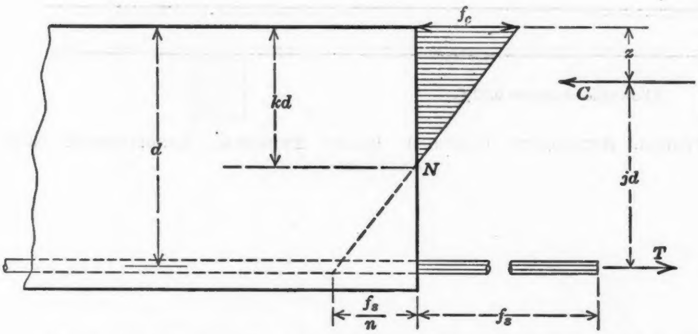

FIG. 1.—NOMENCLATURE FOR CONCRETE BEAM REINFORCED FOR TENSION.

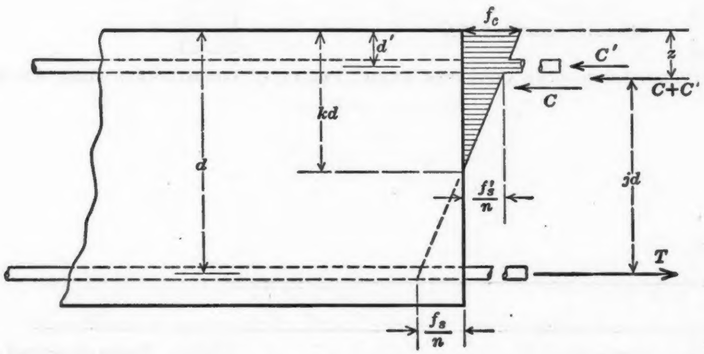

FIG. 2.—NOMENCLATURE FOR CONCRETE BEAM REINFORCED FOR TENSION AND COMPRESSION.

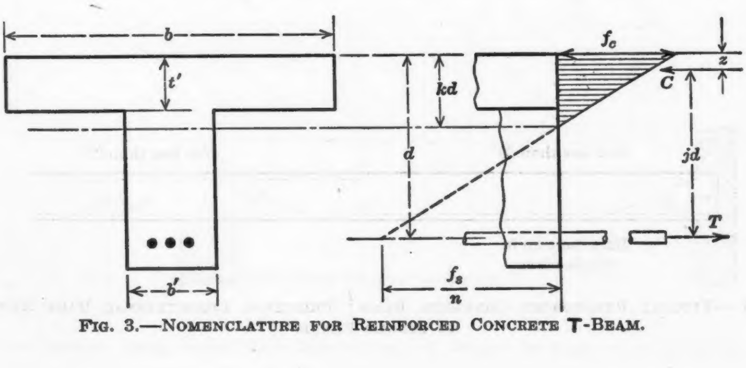

FIG. 3.—NOMENCLATURE FOR REINFORCED CONCRETE T-BEAM.

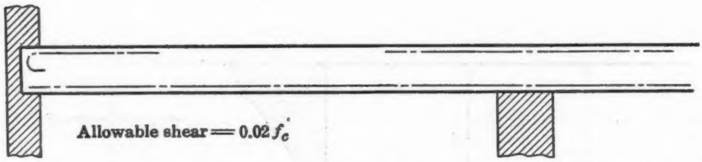

FIG. 4.—TYPICAL REINFORCED CONCRETE BEAM; PRINCIPAL LONGITUDINAL BARS NOT ANCHORED.

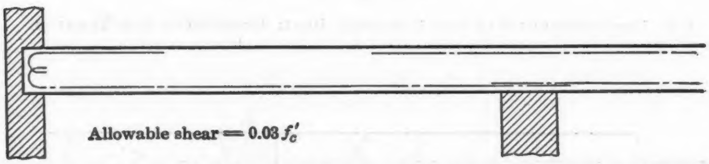

FIG. 5.—TYPICAL REINFORCED CONCRETE BEAM; PRINCIPAL LONGITUDINAL BARS ANCHORED.

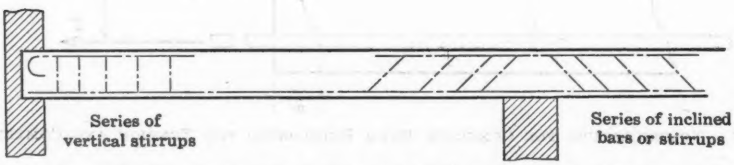

FIG. 6.—TYPICAL REINFORCED CONCRETE BEAM; WEB REINFORCED BY MEANS OF SERIES OF VERTICAL STIRRUPS, OR SERIES OF INCLINED BARS OR STIRRUPS.

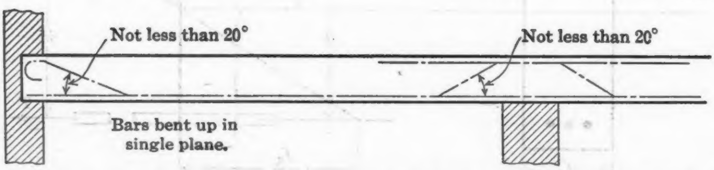

FIG. 7.—TYPICAL REINFORCED CONCRETE BEAM; PRINCIPAL LONGITUDINAL BARS BENT UP IN SINGLE PLANE.

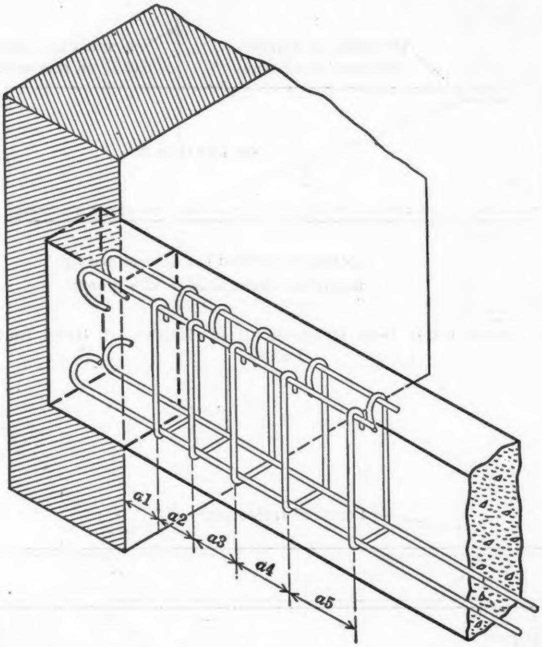

FIG. 8.—TYPICAL REINFORCED CONCRETE BEAM WITH ANCHORED LONGITUDINAL BARS AND VERTICAL STIRRUPS.

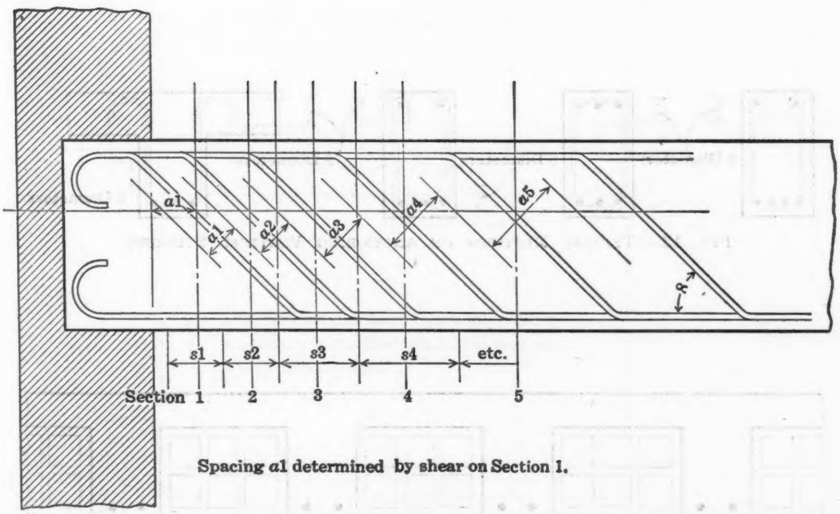

FIG. 9.—TYPICAL BEAM WITH WEB REINFORCED BY MEANS OF SERIES OF INCLINED BARS.

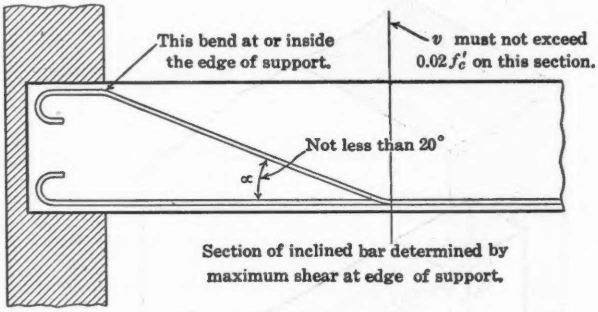

FIG. 10.—TYPICAL BEAM WITH WEB REINFORCED BY MEANS OF BARS BENT UP IN SINGLE PLANE.

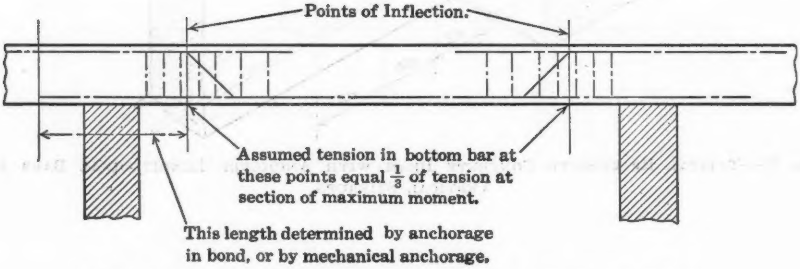

FIG. 11.—TYPICAL WEB REINFORCEMENT FOR CONTINUOUS BEAMS.

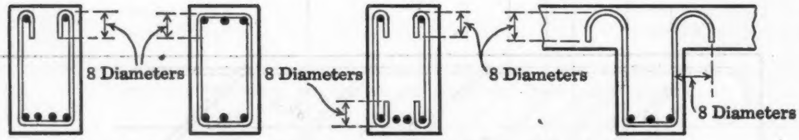

FIG. 12.—TYPICAL METHODS OF ANCHORING VERTICAL STIRRUPS.

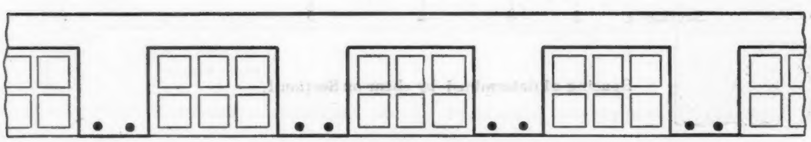

FIG. 13.—TYPICAL REINFORCED CONCRETE BEAM-AND-TILE CONSTRUCTION.

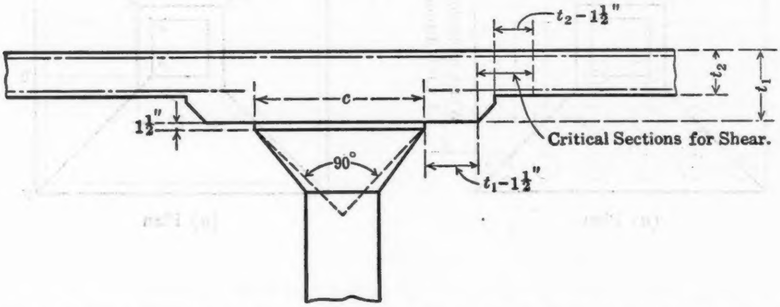

FIG. 14.—TYPICAL COLUMN CAPITAL AND SECTIONS OF FLAT SLAB WITH DROPPED PANEL.

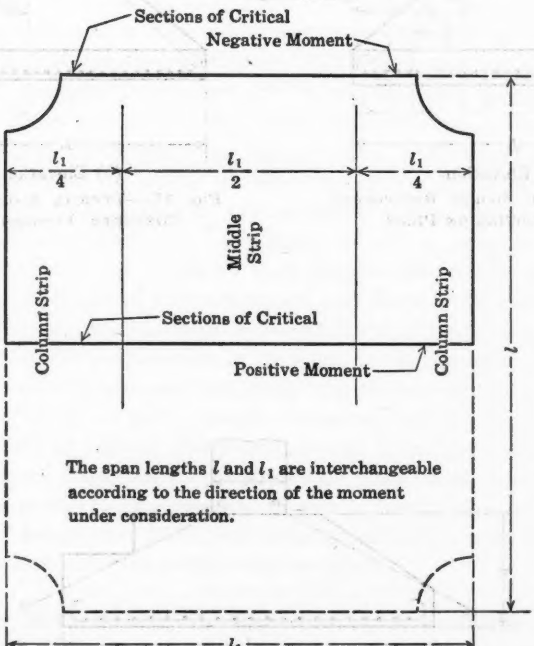

FIG. 15.—PRINCIPAL DESIGN SECTIONS OF A FLAT SLAB.

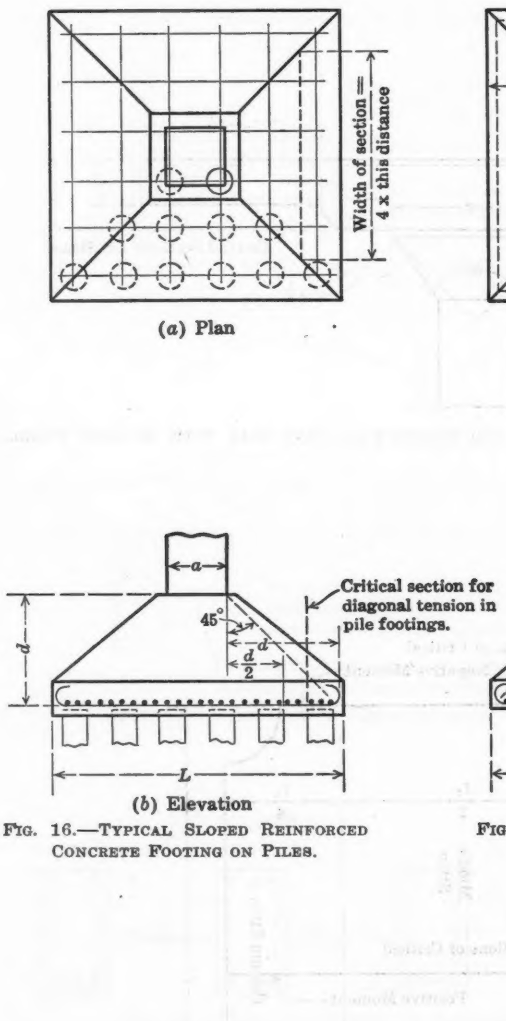

FIG. 16.—TYPICAL SLOPED REINFORCED CONCRETE FOOTING ON PILES.

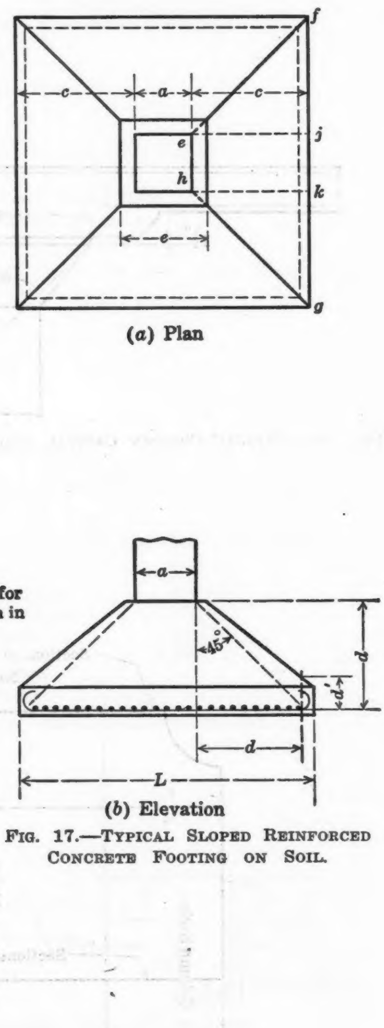

FIG. 17.—TYPICAL SLOPED REINFORCED CONCRETE FOOTING ON SOIL.

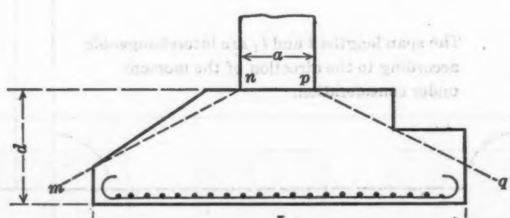

FIG. 18.—TYPICAL SLOPED OR STEPPED FOOTING.

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ODORS AND THEIR TRAVEL HABITS

By Louis L. Tribus,* M. Am. Soc. C. E.

Synopsis.

A famous educator on being asked to explain the term "synopsis" replied by saying that it was an "epitomized exegesis". It is difficult to present a synopsis of this paper, for it is more a rambling collection of notes about "smells", with some incidents illustrating their intensity, prevalence, persistence and movements, than a logical engineering presentation of a definite subject, with conclusions, as a suggested guide to follow. In fact, definiteness as to smells is impossible, even as to any standard of describing them, and the paper makes a point of that fact.

The tolerance of odors given off by essential industries, if due care is taken to reduce offence to the minimum, is considered, the effect of odors upon persons of different susceptibilities is discussed, and the fact that certain races accept as pleasing those odors which produce even disgust in others is noted. Some notes are given in regard to difference of behavior during daylight and night, particularly the curing effect of direct sunlight.

Observed distances of travel of odors are given in numerous instances, and various cases from litigation are reviewed, giving opinions of unnamed experts as to their views upon distances to which odors travel, and the atmospheric conditions under which they become specifically obnoxious.

In general, the types of odors reviewed are those from garbage treatment and sewage disposal works, with some comments upon the production of offensive gases in the preparation of food products, ore and oil refining, etc.

The writer's conclusions are that many engineers ought to give items from their own experiences, so as to provide a reservoir of information that might be helpful in settling various disagreements and even litigation; that chemists might devote more attention to counteractive work; and that some consideration might be given to amendments to public health laws to make possible the stopping of prospective nuisances without having to wait for their actual development before effective action can be taken.

The dweller in the country can generally count on fairly pure air, unrestricted light, good water, and, with a little intelligent care, suitable drainage

* New York City.

and sanitary disposal of wastes. The city or townsman exchanges his freedom from drudgery for a dependence on united efforts to secure the necessities of community life, and encounters thereby many experiences in a lifetime where actual nuisance prevails, detrimental to comfort, if perchance not to health.

Odors (which, when offensive, are usually called "smells") are given off in the conversion of raw materials into food, oils, many metals, gas, and a countless list of articles for human use; and again, through their passage back from such use either to their original or to transformed components.

Some of the objections to such conditions are sentimental rather than strictly justified by nuisance, but the result is the same—vigorous objection giving rise to talk, litigation, and sometimes actual violence.

The personal equation is a strong factor in considering questions of nuisances and their elimination, for there may well be a tolerance bred by acquaintance, or so overwhelming a need for the product, that humanity accepts the conditions without rebellion.

Only recently some of the older children and adults on Barren Island, Brooklyn, N. Y., were asked "What do you most desire?" The response was unequivocal, "Give us back the operation of the garbage plant." With all its atrocious features, it meant employment, money, and upkeep of their homes.

Garbage and Sewage Phases Considered.

This paper presents items chiefly from garbage and sewage phases of the question—for they have come more directly into the writer's purview—and the traveling qualities of odors therefrom, rather than the methods of production.

A point of interest raised, upon which there might be much discussion, is the advisability of making lawful the veto power by Health Commissions or Courts upon projects that practically promise the creation of nuisance, rather than waiting for full construction and actual accomplishment.

So far as "smells" go, there is a wider field for study in counteraction than prevention for; the latter is costly and frequently litigious.

It may be venturesome for an engineer having only an elementary knowledge of the chemical and microscopic constituency of gases and their structure to attempt any characterization of odors (or smells) and their habits of travel. However, having been called at times to make use of nasal, visual and sometimes stomachic susceptibility where questions of nuisance from odors had been raised, the presentation of some notes may be pardoned, in the hope that discussion may bring together more data, crystallization of facts, and agreement as to procedure for the benefit of community welfare.

Mathematical Measurement Impossible.

To begin with, from a mathematical viewpoint the case is hopeless. One cannot say that a particular odor, given off at a specific heat, meeting a certain open air temperature, at a special barometric pressure and known degree of humidity, with a wind blowing just so many feet per second, over a level plain, will be clearly noticeable at a computable distance from its source.

Hence, why should any solution of the case be sought? Simply because odors do travel, and opinions are desired which result very tangibly in settling litigation, where questions of damages and injury to health are at issue.

From an article in *The Lancet* are extracted the following passages:

"If more studied the sense of smell might have been cultivated * * * and the study of odors would have reached a higher plane, though it appears to us the classification of smells must always suffer from the absence of the mathematical analysis capable in optics and acoustics. As it is, in our struggle to describe a smell, we are confined to using such terms as 'peculiar, persistent, pleasing, disagreeable, sour, bad, musty, sickly and so forth'. Now these are all adjectives which give us little enlightenment * * * as compared with color and sound and light mathematical laws. * * * Perhaps sometime smells may be also arranged according to some scale."

The foregoing is quite true; probably six simultaneous observers of six odors, describing them independently, would use not the same six adjectives but at least twelve or fifteen. The real point of importance, however, is the distance that smells travel and their actual effect on human beings.

State Laws Control Nuisances.

State laws usually empower boards of health to control nuisances, nuisances being generally described as "such conditions as offend humanity, either personally or through injury to property, or that may be considered as even detrimental to public health."

Decision is largely dependent on such individual cases as may be in point, and enforcement of restrictions of noisome operations is dependent largely on the locality and the importance to the public of the operation itself. Some trades are essential, yet offensive; they may be tolerated in one place and debarred in another, yet with equal justice.

The second factor of importance brings in the human equation. Some individuals possess a keen sense of smell without effect other than objection to the disagreeable; some are violently affected; others less keenly, although nervously, affected, and in the end seriously so.

The symptoms in most cases are usually loss of appetite, headache, or even nausea.

Some races may not only tolerate but positively enjoy an odor that produces rank disgust in others. All know also that sometimes a whiff of an odor is offensive, while a well saturated atmosphere brings toleration; at other times and more usually the reverse follows, consequently no definite rule could be of universal application.

The Seal and Cod Liver Oil Works of Newfoundland, to the casual visitor, are offensive in the extreme, yet those living near-by become accustomed to the odors, which are typically rancid and more than suggest putrefaction. Fortunately, they do not carry far from the plants, or else become so quickly mixed with the strong ocean air as soon to pass from recognition. They are essential local industries, and hence they are tolerated even if perchance disliked.

Control by Anticipation.

As yet the law does not permit control by anticipation, but actual nuisance must be first produced; then, however, action may be swift and effective. Would it not be wiser, under well safeguarded acts, to permit legal review by boards of health before consent is granted to invest capital, where noisome odors might be anticipated? Of course there are injunction proceedings open to taxpayers, but if State or municipal consent were first required, much litigation might be saved in later complications.

However, legal consents or litigation throw no light on odors traveling, the present point of special interest.

Special Cases Cited.

The writer recalls two cases during his boyhood, one a tomato catsup factory which kept in the open yard several tanks of pulp, just how far advanced in putrefaction and how much was bottled for sale not being of present importance, and the fact is that, although very offensive within 100 ft., the odor could rarely be noted farther away. Operations were closed by order of the health authorities, but on grounds of unsuitable materials rather than general nuisance.

The other case concerned a cream of tartar establishment, where apparently due care was observed in manufacture, yet the vile odors traveled with almost increasingly disagreeable qualities to a distance of at least a mile under almost any atmospheric condition. Here, a shut-down was soon effected on the score of nuisance, though not necessarily directly injurious to public health. The plant was located at the edge of a populous community, and could not be considered an essential industry.

The garbage reduction works on Barren Island (within the Borough of Brooklyn, City of New York) were for years synonymous with stench. Fumes from the retorts and tankage driers passed off in part through the chimneys from which, according to wind and humidity, they traveled 5, 6, or more miles before diffusion and chemical break-up relieved the offense.

These odors, while disagreeable, were not particularly nauseating to most people, and for many years legal actions to end them were not effective. Recently, however, a permanent injunction has been granted. Close to the plant, although intensified, the odors were scarcely more offensive than at considerable distances.

Experience at New Works on Lakes Island.

The great reduction plant built in accordance with many advanced ideas and located on Lakes Island to replace the Barren Island works, for the several months of operation prior to its being closed as a nuisance, gave off gases that persisted in nauseating quality and strength to points 8 and 9 miles away. Rather curiously, although twenty-four hour operations were maintained, these peripatetic smells became more offensive after sundown. Evidently, the sun's rays possessed a deodorizing power which was joyfully welcomed by the burdened population through those months of torment.

Years ago it was discovered, practically by accident, that sunlight streaming into tanks holding tried out seal oil, while going through the curing process, killed in large measure the exceedingly vile odors of decomposition, and thus made life endurable for the workmen.

The odor problems of many cities, aside from offensive trades, are those from garbage works and sewage disposal systems. A great many reams of paper have been covered with lawyers' briefs and thousands of witnesses have been examined in trying to solve the difficulties due to noisome smells; individuals attacking communities and private corporations, and communities attacking each other.

New York Case Against Bayonne, N. J.

A long drawn-out action was that of New York State *vs.* New Jersey, trying to enjoin the passing of corrosive gases and offensive odors from certain great manufacturing plants in Bayonne, N. J. While no summary closure was effected, a very great amelioration was secured. A rather noteworthy step was taken by some of the defendants in gathering the worst gases into two great chimneys, and passing them into the air at a height of 360 ft. or so above ground. While the eye can see the output drifting for several miles, diffusion and perhaps transformation largely occurs before the odorous gases descend to ground level again, unless a strong wind blows them earthward to produce their old-time results of throat and nose irritation (fumes from copper smelting predominate). Such fumes quickly kill vegetation.

Another bitterly resented crop of fumes came from great manufacturing establishments, again along the New Jersey shore, at Edgewater. In spite of the mile of Hudson River water between the shores of what is virtually a great canyon formed by the high banks, residents along Riverside Drive, New York City, and other neighboring streets, were driven almost frantic at times by the peculiar and irritating gases given off in the preparation of certain foods, chemical, and other commercial products.

Here again litigation and pressure have forced great changes for the better, but the point of present importance is not the peculiarities of litigation or the fact that usually such pressure brings some relief, but rather that odors do travel. Of still greater importance is the fact that such odors can usually be anticipated and controlled.

Gas Manufacture Now Inoffensive.

The earlier manufacture of illuminating gas was attended by odors from the wastage of the first output of the retorts and from the usually open tanks of coal-tar and other by-products. Unless disseminated by strong winds these odors did not as a rule make themselves known at great distances, rarely more than ½ mile, but much litigation ensued nevertheless, particularly in Great Britain. At the present, with the use of oil so largely replacing coal in illuminating gas manufacture, there is less of the waste from retorts, and the by-products, having commercial value, are conserved at once for sale.

As a consequence, gas-making has nearly left the ranks of offensive industrialism, barring occasional accidents or carelessness.

In considering odors which give offense to communities, those from fuel consumption are largely ignored, for when serious in nature, it is more because of mixture with unconsumed carbon in the smoke, rather than the irritating or noisome constituents themselves; furthermore, dwellers in cities are so thoroughly saturated with ordinary coal gases as to have largely passed the stage of noticing them, unless they are present in great excess.

Soot as a Nuisance.

The soot elements have been productive of many a strenuous fight, and will still continue to do so until that day comes when, with power plants at the coal mines, electricity will be developed to do much more of its cleanly work in the cities, with the tremendous saving in labor, space, wastes, etc. The imagination could well run riot over the practical advantages of electric power, and with only a trifling betterment in mechanical conditions, the ultimate economies also. Of course, water power converted to electricity will produce the same result.

A public commission in Idaho, not long ago, issued its opinion, after obtaining much testimony, that electricity cannot take the place of the direct use of coal for heating; another body in Ontario came to a somewhat similar conclusion. Positiveness, however, is dangerous and the "cannot" of to-day becomes the "accomplishment" of to-morrow.

Odors Carried Long Distances.

Long before land can be seen from the ship, the peculiar and rather pungent odors of tropical vegetation in the West Indies can be discerned, indicating their travel of 25 or 30 miles at least. The ocean waters themselves may be somewhat responsible, due to stream flow and land washings, but it is atmospheric carriage in largest measure.

That leads to another phase of the subject, where possible absorption of gases by water, with movement to new scenes of activity before final transformation or diffusion of obnoxious elements may tend to a later separation, again of gaseous products, and to the extent of causing a transplaced nuisance. This has been often noted in sewage cases, where volatile elements from treated sewage effluents seem to have been carried in actively flowing streams for considerable distances.

In such cases, however, unless atmospheric conditions are very favorable, these finally liberated gases do not usually travel far from the stream itself. Even in the case of the lower Passaic River, Gowanus Creek, and formerly the Chicago River, the well-known trio of unenviable waters charged to practical saturation with sewage, the odors were not noticeable usually for more than a few blocks away from their banks.

From a recent report of the Albany Sewage Disposal Plant, the following statement is quoted:

"There is practically no nuisance caused by odor from the plant, as it has never been discernible for a greater distance than 300 yd., and then only on muggy summer nights."

Odors from Garbage Treatment and Sewage Disposal.

Returning, however, to the fields in which many communities have a common interest in either providing against prospective nuisances or in curing them when created, namely, garbage transportation and treatment and sewage disposal, a brief word on processes may not be amiss.

In the treatment of garbage, aside from field spreading, three distinct methods are in vogue, each with its strong advocates, and each, of course, with its various modifications:

(*a*) High temperature incineration (1 300° to 2 200° Fahr.) where, under proper operation, no offensive odors pass from the plant except that a slight "caramel" aroma may at times be noted in the fleecy, whitish smoke that comes from the chimney, but scarcely objectionable except to the ultra-sensitive. We do not speak of esthetic objections, but rather of sensatory causes.

(*b*) Reduction by cooking, according to one or another of several processes. To date, except from perhaps very small scale operations, odors are given off from all practicable plants, having quite distinctive characteristics of aroma, persistence in staying qualities, and decided diligence in travel. These have already been referred to in the cases of the Barren Island and Lakes Island plants, of different types; the same conditions prevail at Boston and Cleveland, and perhaps not quite so noticeably, yet actually, at Columbus and Chicago. In the latter case, the situation is such that other kinds of odor-producing establishments are near-by, so that any one is less conspicuous.

(*c*) Transformation by hog feeding. This became during the World War a popular and intensive process and a profitable one, but less so now that the price of pork has dropped back to nearly old-time rates. Properly managed it may not contribute largely to community smells, but in olden days the reputation of hog farms was not enviable, their sweetish and swill odors being distinctly noticeable at considerable distances.

The Lakes Island Case in New York.

The Lakes Island (New York City) reduction case, being quite recent and very hotly contested, might be reviewed at some length, to indicate the lines of thought developed and opinions brought out. It progressed in three stages, as follows:

First.—The objection of residents of the Borough of Richmond, New York City, to having erected within its confines a plant planned to reduce some 2 000 or more tons of garbage a day produced by three of the other boroughs, entailing also the transportation of such garbage on scows closely paralleling for five miles the course of the Municipal Ferry which carried about 45 000 passengers daily, and passing along and within a few hundred feet of some ten miles of Staten Island's water-front.

The State Commissioner of Health held a long series of hearings, calling to sit with him a Professor of Sanitary Engineering of Harvard University and a Deputy State Attorney General. The case was conducted by the District Attorney of Richmond County for the protestants; by an Assistant Corporation Counsel of New York City (defending the contract), and by counsel for the Metropolitan By-Products Company, the prospective erectors and operators of the plant. A great deal of testimony was taken, the hearings extending over many weeks, resulting in a report to the Governor that a nuisance would probably develop, but that until such actually occurred, the State law would not permit action. The Governor instructed the Commissioner of Health to keep close watch, and directed the District Attorney to report also to him at the first indication of trouble, so that he might take summary steps for abatement.

Second.—The residents of Richmond were not satisfied, even though having full confidence in executive intent, and consequently a Special Grand Jury took cognizance of the case and through the District Attorney and special witnesses, became advised of many facts, but again found that limitations of law prevented effective action. The construction of the plant had progressed, however, nearer to completion and early operations fully verified anticipation, that while perhaps possible to operate without offense, it was not practicable so to do. A presentment but no indictment followed.

Third.—The New York City Health Commissioner took a hand, operations on nearly full scale having been undertaken, and after hearing much testimony, expert and lay, very quickly decided that the nuisance was actual, intolerable, and unjustifiable, and forthwith forbade further operations. Combined with this official "order" came bankruptcy of the Company, and the residents were relieved of their troubles.

The odors were of nauseating quality and were seriously offensive at distances of at least eight miles from the plant. The discriminating nose could detect three characteristics in the stench; that of caramel or burning vegetable matter, semi-rancid decomposing swill (combined vegetable and animal elements), and of the solvent (a chemical midway between gasoline and kerosene).

These odors would settle in topographic and atmospheric pockets, and treated the public to many surprises as to their lasting qualities and intensity. The effect on different systems was quite diverse, producing severe headaches in many, nausea in others, lassitude in some, and generally violent wrath in all.

Varying Opinions of Experts.

In the two earlier proceedings the opinions of experts varied widely, several testifying for the Company. The City took the position that odors of rotting garbage, conveyed on open scows, if covered with tarpaulins, could not escape and would not be noticeable more than about 100 ft. away, at worst. Others stated that from the plant built and operated as promised there could be no escape of gases to annoy even visitors to the property.

Theoretically, the last condition should have been true, except for the unloading of the partly decomposed materials. Practically, of course, no

such operations would be carried on as they should be, and hence the conditions that actually developed should have been anticipated, in part at least.

There was much testimony offered as to the prospective and real qualities of the gases to be produced, and as to those actually emanating from the plant. This testimony tended, in weight of opinion, to support the claim of their being detrimental to health, even if not actually or violently poisonous.

There could be no divergence of unbiased opinion as to their injurious effect on real estate values, and as the loaded scows in service were not well covered, all theories as to their agreeable nature were "knocked endwise."

Details of Methods.

As to process, the following may be helpful to an understanding of the situation. The scows holding from 300 to 500 tons of garbage from 2 to many days old were towed from the Bronx, Manhattan, and Brooklyn to and alongside an open wharf at Lakes Island along the west coast of Staten Island, Borough of Richmond, City of New York. Grab buckets lifted the mixture to hoppers from which belt conveyors carried it to the double kettles holding 5 to 6 tons each. Live steam, introduced into the space between the inner and outer kettles, cooked and vaporized the constantly stirred mass. As the watery vapors passed off through a check-valved steam pipe system, solvent was pumped in to digest the grease. After 8 to 12 hours treatment, the dissolved grease was drawn off for settling and barreling. The vapors were supposed to be condensed for recovery of the surplus solvent, and the fairly dry tankage was conveyed to the storehouse, first being roughly screened, to take out such bones and rags as had escaped the pickers while the raw goods were on the first belt conveyors. Cans, glass, table silver, paper, and a multitude of articles were taken out by the human pickers in that stage of the work.

All processes where heat would develop odor were supposed to be closed, but carelessness, leaks, and failure in some features of the plant, permitted the escape of smells that, as a stench, brought catastrophe.

Monticello Sewerage System.

Another recent instance of litigation over odors has grown out of the operation of the Monticello sewerage system. About ten years ago, that village called on the writer's firm to modify, if necessary, and complete the system that had been well started, but whose designing and supervising engineer had died.

As completed, the disposal works occupied a somewhat swampy area of rather tight soil and consisted of three covered sedimentation tanks, four open stone contact beds, and two large open underdrained sand filtration areas, the processes being successive, and operation alternating. The effluent from the underdrains, and from the sludge bed (used about once a year) passed into a brook which a few hundred feet away united with another stream that carried the village street drainage, and a smaller one that also first passed partly through the village.

Tannery sewage, after going through its own detention septic tanks, entered the sewerage system, but apparently did not materially interfere with bacterial action in the disposal works, although it added at times elements of color from waste dyes.

At a distance of ½ mile from the works the combined streams crossed a main highway, near which was a summer boarding house. The owner brought suit for losses of income, charging that odors from the disposal works had driven away the boarders.

The normal effluent from the plant was clear, practically colorless, and almost odorless. The odor could scarcely be noted 50 ft. away, and along the flow line of the brook soon became so mingled with the swamp smell as to be indistinguishable. At times, however, some carelessness of operation or accident, or flood conditions, carried an overflow of liquid from the sand beds into the brook, this liquid being the result of two processes of bacterial break-down. Also, under certain atmospheric conditions, while the twice treated sewage was standing on the sand beds, slowly percolating to the underdrain, odors rose and sluggishly moved over the nearly level valley, then combining with the plaintiff's own cattle and hog yard smells and the natural swamp vapors, produced an unpleasant effect.

A great deal of testimony was taken in the many days given to the trial, tending to substantiate the claim that sewage odors did at times travel the ½ mile or more. Whether such fact could carry financial damages was solely within the province of the Court to say. It was clearly brought out and acknowledged that the plant itself was suitable, and properly installed.

The consensus of testimony was to the effect that atmospheric conditions as to humidity or dryness largely affected the lasting and persistent nature of the odors, and that light winds would carry them to considerable distances, while heavy winds would cause rapid diffusion and soon put them "out of business."

The Court has recently rendered rather an unique decision, awarding $1 200 damages in the $25 000 claims, divided $400 payable by the village, $400 by unknown offenders, and $400 by the plaintiff himself as a contributor to the trouble. The village is also directed to enlarge its plant, so as to more adequately care for the maximum sewage load.

General Testimony on Odors.

Writers have in general expressed themselves rather conservatively, as a rule, when treating the subject of odors, for except when personal experience is available, little that is conclusive has been brought out.

One well known treatise on sewage states that "odors of aerobic treatment are less offensive than from anaerobic." In other words, open-air oxidation gives off less putrid gases than closed bacterial methods. In another passage the same treatise states "fresh sewage on trickling filters is not noticeable over 100 yards away * * *, from septic tanks some one-half mile." Another writer says:

"The odors which arise from decomposing sewage do not cause typhoid fever, or smallpox, or scarlet fever, or any other contagious disease. They may, and often do, reduce the comfort and happiness and, consequently, in a sense, the health of the people subject to them, but it cannot be maintained that they cause serious sickness."

From the testimony of experts in the Lakes Island case, the following brief sentences are quoted:

"Odors travel greatly under light air and high humidity."
"Garbage scow smells are noticeable from one-third to one-half mile."
"Odors travel 3 to 5 miles, increasing with quantity, * * * night travel is more offensive than day."
"Odors carry, say, three miles."
"Odors not so offensive with high winds."
"Barren Island odors are from the drying plant."
"Garbage odors will travel long distances."
"Could not smell plant (garbage) over 500 ft."
"Odors would not carry far."

Two of the Southern States a few years ago fought out very conclusively the principle that people are entitled to fairly pure air, as well as pure water; in that case, the fact of the travel of odors to an offensive degree and unreasonable distance was well established.

Recent Developments.

The phase of the subject considered in this paper is more that of odors which are offensive to people, than gases which cause damage to property. The principle is becoming more and more firmly established that upon the party giving the cause of annoyance devolves the duty of relieving it. In matters of sewage outflow into, and water supply from, the same stream there is a duty charged upon both interests, first to mitigate offense, and second, to prevent danger; the right of drainage is held to be practically equal to the right to a pure water supply, so that treatment works are required to be operated by both parties.

At one time there was a great hue and cry about slaughter-houses in cities, and the offensive smells that came from them and passed liberally through the neighboring community. Many an action was brought to close them, but as a rule the Courts determined that the service was needed by the public, and Boards of Health were instructed to keep very sharp supervision to prevent needless annoyance.

Later, as greater demand developed for cleanliness, slaughter-houses have gradually almost ceased to be offensive. Economic questions also came into the case, for what had once been wasted and allowed to rot and smell, until sporadic house-cleaning was indulged in, became of value as commercial by-products. Blood once passing into sewers goes into feed and fertilizers; hair, hide, bones, horns, etc., all have market uses, and the offal in some measure finds a sale; even considerable imports are brought to the United States of the entrails of animals killed in Japan and China.

To attempt to analyze an odor is hopeless, though it is certainly some form of emanation that has motive power of its own, but when sufficiently

diffused is broken up into elements that cease to have original "personality", yet so long as traces last some characteristics persist.

The writer has visited many garbage treatment works and has noticed the clinging nature of the odors, which have often been retained by the clothing 12 or 15 hours after leaving the plant; this being more noticeable from reduction than from incineration works. This is not indicative so much of traveling capacity, as of the simpler or more homogeneous form of the smell atom; it therefore does suggest a persistent character, hence the ability to make itself known at great distances from the source.

Experiments have been made as to the toxic effect of garbage odors, but not conclusive enough to warrant much discussion.

In the manufacture of linoleum and oil cloths various oils are used which, in changing to fixed forms, give off smells that permeate the air for sometimes several square miles around the plant. The preparation of linseed oil also produces smells that carry considerable distances. In neither of these cases, however, is there a condition that seems to develop unhealthful reactions in humanity.

From the foregoing and similar facts it may be asked, "Should and can odors be classified by any scale of descriptive terms? Can odors be analyzed to discover the atoms that make them agreeable or offensive, and thus find what other atoms may be needed and used to neutralize their influence?"

A homely instance is the cooking of cabbage and its filling a house with the characteristic and not altogether agreeable smell; a little soda in the water in which the cabbage is cooked practically neutralizes the odor without injurious effect. Parasites are grown to kill off other species detrimental to crops. Antidotes for many things that cause trouble are used.

There are, of course, well known germicides, but these are of greater service in arresting or counteracting putrefaction, than in transforming the gases already developed, unless to mix with them and by combination change their identity.

The work to be done may lie more in the field of the chemist, but the causes of many odors occur in the field of the engineer, so that from his knowledge and experimentation also there may come a solution for many a community's fears and discomfort, and prevention of the development of many an actual annoyance.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

DAVID HERBERT ANDREWS, M. Am. Soc. C. E.*

DIED FEBRUARY 24TH, 1921.

David Herbert Andrews was born on September 17th, 1844, at Pepperell, Mass., where his father, the Rev. David Andrews, was minister of the Congregational Church.

In early life, Mr. Andrews, being inclined to mechanical pursuits, found employment in the machine shops of Fitchburg and Worcester, Mass. Believing that a college education would help him in his chosen line, he prepared himself by night study to enter the Chandler School of Science and the Arts connected with Dartmouth College, Hanover, N. H., and was graduated therefrom in 1869 with the degree of Bachelor of Science. In 1908, the Honorary Degree of Master of Arts was conferred on him by that institution. In 1896, he was made a member of the Board of Visitors of the Chandler School Foundation, which position he filled until his death.

After his graduation, Mr. Andrews taught Mechanical Drawing in a night school at Fall River, Mass., and, later, in 1871, entered the service of the National Bridge and Iron Works, at East Boston, Mass., as Engineer. In this capacity he began a career notable for its part in developing the art of construction in iron and steel, at that time in its infancy. As one of the pioneers in this line, his natural talent for invention, his painstaking thoroughness, his industry, good judgment, and courage, helped him greatly in entering a field that was new to the Profession, using new materials, requiring new machines for fabrication, and new methods and appliances for erection. These pioneers were called on to design as well as to build, and they developed the theory as well as the practice of Structural Engineering in the United States.

About 1872, the National Bridge and Iron Works built the train-shed roof at its Boston Terminal for the Boston and Lowell Railroad Company. This roof consisted of braced arches of 116 ft. clear span and 63 ft. clear height at the crown, with framed purlin trusses and I-beam rafters. After forty-eight years of service, it was removed in 1920. The design was made by Mr. Andrews and, as the graphical method of determining stresses in braced arches had not been developed in this country at that time, he devised a scheme for making the necessary computations by studying the effect of a system of weights hung over sheaves on a catenary chain. The building of this roof caused considerable interest in engineering circles at the time, and the structure when built was the most noted of its kind in the country.

The National Bridge and Iron Works met with financial difficulties, and, in 1876, its affairs were wound up by Receivers. Mr. Andrews bought such tools and machinery from the Receivers as he deemed advisable, being

* Memoir prepared by J. P. Snow, M. F. Brown, and John C. Moses, Members, Am. Soc. C. E.

obliged to borrow most of the capital therefor, and set up a plant, under the name of the Boston Bridge Works, at East Cambridge, Mass. As sole proprietor of the enterprise, he proved himself a business man of great ability, soon achieving financial success as well as a wide reputation for the good workmanship of his structures, a large proportion of which were also designed by him and his engineers. This reputation and success was founded on his industry, strict attention to business, and to the confidence that he inspired in every one—employees, capitalists, supply firms, and customers—of his entire reliability and strict integrity.

A list of the work done during the first twenty years of his company's existence would comprise most of the railroad and highway bridges of New England and a considerable number in other parts of the United States. The first steel buildings in Boston would also be included. Mr. Andrews' talent for invention produced the first derrick car for bridge erection, in the early Eighties, and a new type of locomotive turntable which is still in wide use. In 1896, his plant was completely destroyed by fire, but, not discouraged, he immediately started plans to rebuild, and after personally making a study of the best plants in the country, he designed his new factory and supervised its construction in every detail. In 1901, he incorporated The Boston Bridge Works, continuing as its President until his death and being succeeded in that office by his son, John G. Andrews.

Mr. Andrews' professional and business life covered a period of fifty years, during which time the use of iron and steel increased vastly. It required courage and industry, technical skill, and practical common sense to succeed in the early days, and this record is made as an example and encouragement for those who now take up the work that has been begun for them.

Mr. Andrews served as Chairman of the Chelsea School Board during his residence in that city. He moved to Newton Center, Mass., in 1890 and took an active part in the church and civic affairs of that community. He was elected a member of the Boston Society of Civil Engineers in 1881, and was a member of the Engineers' Club of Boston. He was also a member of the American Society of Mechanical Engineers, the Boston City Club, and of other civic and social organizations.

Mr. Andrews was married in 1872 to Miss Clara Gilbert, of Concord, N. H., who survives him, together with a daughter and three sons, two of the latter having been long associated with their father in his business.

Mr. Andrews was elected a Member of the American Society of Civil Engineers on September 2d, 1885.

GEORGE PIERREPONT BLAND, M. Am. Soc. C. E.*

Died April 18th, 1921.

George Pierrepont Bland was born in Roxborough, Philadelphia, Pa., on December 30th, 1851. He received his early education at the public schools

* Memoir prepared from information furnished by Dr. Henry S. Drinker, President Emeritus, Lehigh University, South Bethlehem, Pa., and on file at the Headquarters of the Society.

of that place, after which he entered Lehigh University and was graduated from that institution, with the Class of 1872, with the degree of C. E.

Immediately after his graduation from Lehigh, Mr. Bland entered the employ of the Pennsylvania Railroad Company, under the late Joseph M. Wilson, M. Am. Soc. C. E., then Engineer of Bridges and Buildings, and was engaged in office work incidental to the design, erection, etc., of the bridges used on its main line and branches.

In 1876, Mr. Bland accepted an appointment as Engineer for Cofrode and Saylor (The Philadelphia Bridge Works), a bridge-building firm, at Philadelphia, Pa. Subsequently, he entered the general contracting business under the firm name of Gibson and Bland which, later, became Bailey, Milliken and Bland.

In 1895, Mr. Bland organized and established the Keystone Structural Company, the fabricating shops of which are located at Royersford, Pa. As President of this Company, he had offices in Philadelphia where he attended to the making of contracts and the designing and detailing of structural steelwork for buildings, bridges, etc., continuing as the active head of the business until his sudden death from angina pectoris on April 18th, 1921.

Mr. Bland was married on February 3d, 1875, to Miss Alice A. McCalla, who, with one daughter and two sons, survives him.

While at Lehigh he was noted as one of its most brilliant students and was generally liked and esteemed. As an alumnus, he was always interested in University affairs and responded liberally to calls for support and aid.

Mr. Bland's tastes were markedly scientific, and during his life he was rarely without some mathematical work which he used mainly as a recreation from professional and business cares. Although he was a good executive and managed his business with care and skill, he was primarily a student, which trait had developed more and more during the later years of his life.

In disposition, Mr. Bland was retiring, and although he was sufficiently aggressive to conduct his business successfully, he was by no means what is called a good "mixer"; after his early friends and comrades had passed away, he did not easily form other friendships, and his new connections were usually of a business nature.

Mr. Bland was elected a Junior of the American Society of Civil Engineers on April 7th, 1875, and a Member on May 4th, 1881.

ALFRED PANCOAST BOLLER, M. Am. Soc. C. E.*

Died December 9th, 1912.

Alfred Pancoast Boller was born in Philadelphia, Pa., on February 23d, 1840. His early education was received at the Episcopal Academy of his native city, followed by his graduation, with the degree of A. B., from the University of Pennsylvania in 1858. He then entered Rensselaer Polytechnic Institute from which he was graduated with the degree of C. E., in the Class of 1861. It is interesting to note that among his classmates at Rensselaer

* Memoir prepared by S. Whinery, M. Am. Soc. C. E.

were Estevan A. Fuertes, M. Am. Soc. C. E., William L. Haskins, Robert Neilson, M. Am. Soc. C. E., T. Guilford Smith, M. Am. Soc. C. E., and William N. Simmington, all of whom are now dead.

Soon after his graduation, Mr. Boller began his engineering career as a Rodman on the Nisquehoning Railroad, advancing to the positions of Instrumentman and Topographer. In the latter capacity, he made an elaborate survey and map of the middle and southern anthracite coalfields for the Lehigh Coal and Navigation Company, and, in 1862, was employed on the repairs to the Company's Canal, following the great flood that damaged and destroyed parts of it.

In 1863, he entered the service of the Philadelphia and Erie Railroad Company, in the Department of Bridges and Buildings, and from this period he turned his attention chiefly to structural engineering. During this engagement he was detailed to inspect and report on the harbor facilities of various cities on the Great Lakes, with reference to proposed harbor improvements at Erie, Pa.

In the summer of 1865, Mr. Boller was in charge of the construction of a suspension highway bridge at Williamsport, Pa. In the spring of 1866 he was appointed Engineer of Bridges on the Atlantic and Great Western Railroad and, during this engagement, planned the Cattaraugus Viaduct and an international bridge over the Niagara River at Black Rock, N. Y. After the failure of this railroad enterprise, he became, in the fall of 1866, Chief Engineer of the Hudson River Railroad, but resigned after about six months to associate himself with Samuel Millikin as Agents of the Phœnix Iron Company. During the four years of this partnership, Mr. Boller was connected, either as Engineer or Contractor, with a number of enterprises, among which were the Bridgeport Bridge, the construction of Piers 38 and 39 on the North River, and the design of the St. John's Park Station, of the New York Central and Hudson River Railroad, in New York City.

In the summer of 1871, he accepted the position of Vice-President and Engineer of the Phillipsburg Manufacturing Company, engaged in the design and construction of bridges and other structural iron work, serving in that capacity until its failure in the panic of 1873. During his connection with this Company, he designed and supervised the construction of a number of railroad and highway bridges and other structures, among the former being the Park Avenue Bridge over the Morris Canal at Newark, N. J., which had some novel features, described in a paper presented before the Society.* Following the collapse of the Company, he served as Chief Engineer of the Manhattan Elevated Railroad, the Yonkers Rapid Transit Commission, and the West Side and Yonkers Railroad.

In 1874, he opened an independent office in New York City and soon acquired a large and important professional practice, which he continued until his death, which occurred at his home in East Orange, N. J., on December 9th, 1912.

Among the more notable enterprises on which Mr. Boller was engaged may be mentioned: the Staten Island Rapid Transit Railroad; the New York,

* *Transactions*, Am. Soc. C. E., Vol. II (1873), p. 379.

Providence and Boston Railroad; the Thames River Bridge, involving difficult and novel foundation work and approaches; Consulting Engineer to the Department of Parks and to the Department of Public Works, New York City; to the Rapid Transit Commission (New York City) of 1884; to the contractors for the foundation of the Statue of Liberty, New York Harbor; the Albany and Green Bush Bridge; the West Side and Yonkers Railroad, including the Eighth Avenue Bridge over the Harlem River, New York City; the bridges for the New York Central and Northern Railroad over the Bronx River and at Croton Lake*; the Congress Street Bridge at Troy, N. Y.; the Eastern Avenue Draw Bridge at Boston, Mass.; the Harlem River Bridge at 155th Street and Seventh Avenue (more commonly known as the Central or Macomb's Dam Bridge), New York City; the Arthur Kill Bridge, New York City; and the bridge over the St. Louis River, at Duluth, Minn. Mr. Boller was also a member of contracting firms for the construction of the Bergen County Branch of the Erie Railroad, and the foundations for the great gas tanks of the Bay State Gas Company, at Boston, Mass.

For a number of years, he was the Consulting Engineer for the various projects and improvements of the Lake Superior Company, at Sault Ste. Marie in Canada and Michigan. He was retained, in a number of cases, as Consulting Engineer by the United States Government and by the State of New York. He also acted as Consulting Engineer on a number of very deep and difficult building foundations in New York City.

In 1898, Mr. Boller formed a partnership with the late Henry M. Hodge, M. Am. Soc. C. E., under the firm name of Boller and Hodge and, in 1912, Howard C. Baird, M. Am. Soc. C. E., who had been connected with the work of the office for some time, was admitted to the firm, which then became Boller, Hodge and Baird, Mr. Boller continuing as the senior member.

Under the first named firm, the bridge over the Arkansas River at Little Rock, Ark., was designed and built, as well as the important bridge over the Monongahela River and that over the Ohio River at Mingo Junction for the Wabash Railroad, at Pittsburgh, Pa.; and the bridges for the Algoma Central Railroad in Canada, the Municipal Bridge over the Mississippi River at St. Louis, Mo., the State Bridge over the Connecticut River at Saybrook, Conn., and a number of bridges for the City of New York, were designed. In 1907, the firm was appointed as Consulting Engineers to the Joint Commission of the States of New York and New Jersey to report on plans for interstate bridges over the Hudson River at New York City. The firm also acted as Consulting Engineers on the steel framework for the Singer Building and the Metropolitan Life Building, in New York City. During this period, Mr. Boller served as Consulting Engineer on the new masonry arch bridge at Hartford, Conn., which was completed in 1907, and is commonly considered to be the finest example in America of the stone arch bridge.

Under the firm of Boller, Hodge, and Baird, construction work on a number of the previously named structures, including the Municipal Bridge at St. Louis, was prosecuted, and the bridge over the Connecticut River at East Haddam, Conn., and the concrete bridge over the Hillsborough River

* *Transactions*, Am. Soc. C. E., Vol. XI (1882), p. 150.

at Tampa, Fla., were designed and built. Mr. Boller and his firm acted as Consulting Engineers for the National Railways of Mexico and designed bridges for railroads in South and Central America, Cuba, the Philippines, and Haiti, as well as many minor railroad and highway bridges in the United States.

Mr. Boller was recognized as standing in the front rank of structural engineers and as an expert in bridge engineering. Familiar with the technical side of the art and science, he was especially noted for his practical good sense and sound judgment. Not a few of his bridges were characterized by their originality and boldness of design. The draw-span of the Thames River Bridge, 503 ft. in length, and weighing 1 200 tons, was the longest attempted until that time. Another remarkable structure, the great viaduct (or Central Bridge) over the Harlem River at 155th Street, New York City, 4 500 ft. long, costing over $2 000 000, and having a draw span weighing 2 400 tons, was stated at the time to be the heaviest movable mass in the world. The channel span of the Municipal Bridge over the Mississippi River at St. Louis, Mo., was at the time it was designed the longest fixed truss span in the world, 670 ft. between centers of piers. To the great advances made in bridge design and construction during his professional life, Mr. Boller contributed his full share. He was the author of a small book, published in 1876, entitled "Practical Treatise on the Construction of Iron Highway Bridges, for the Use of Town Committees." This book was chiefly intended for the information and assistance of municipal officials in that line of their duties.

Mr. Boller was especially well qualified by natural endowments, culture, and training for his vocation. His mind was keen, logical, and persistent. His judgments and opinions were the result of careful and deliberate consideration, rather than of hasty impulse or impression. He had an intuitive sense of, and fondness for, the artistic, and one of his leisure diversions was the sketching in water-colors of landscapes, at which he was notably successful for an amateur. His appreciation of architectural symmetry had a marked influence on his bridge designs, his constant effort being to combine technical principles and practical utility with symmetrical and pleasing outlines. Notable examples of his success in these efforts are the 155th Street Bridge in New York City and the Hartford Bridge.

He was married on April 28th, 1864, to Katharine, daughter of William Henry Newbold, of Philadelphia, Pa. Mrs. Boller and three sons and two daughters survived him.

Mr. Boller was an exemplary citizen. For fifty years, his home was in a suburb of New York City which grew, in that time, from a mere village to an important suburban city, and in the inception and development of its public works and civic enterprises he took a keen interest, and his counsel and aid were always sought and freely given. As the first President of the first Shade Tree Commission in the State, he organized it on an efficient and successful basis. Few civic organizations or enterprises were considered complete without his name and influence. For a long period he was a member and influential Vestryman of Grace Protestant Episcopal Church, one of the leading churches of the vicinity.

In his relations with his fellow engineers and associates, Mr. Boller was courteous, considerate, and helpful, particularly to the younger members of the Profession, many of whom remember with gratitude the aid and encouragement extended to them.

In business matters he was diligent and efficient, and especially careful to be just and fair to every one.

No term seems so fittingly to describe his personal character and social qualities as the word lovable. His fondness for, and his devotion to, his home and his family were notable traits in his character.

He was active in the organization of the American Institute of Consulting Engineers, serving two terms as its first and second President. He was also a member of the Institution of Civil Engineers, and belonged to the Century Club and to other clubs in New York City and vicinity.

Mr. Boller was elected a Member of the American Society of Civil Engineers on December 4th, 1867, soon after its resuscitation and reorganization, following the Civil War. He served as its Secretary, without compensation, in 1870-71, as a Director in 1872, and as a Vice-President in 1911-12, his death occurring during his term of office. He was a member of the Special Committee on the Means of Averting Bridge Accidents (1873) and of the Special Committee on Steel Columns and Struts (1909). He took an active interest in the affairs of the Society, contributed several papers to its *Transactions,* and frequently took part in the discussion of professional subjects.

Mr. Boller's long and busy life exemplified the best traits of American manhood and citizenship. As an engineer, not many have contributed more to the usefulness, dignity, and honor of the Profession.

ISAAC WENDELL HUBBARD, M. Am. Soc. C. E.*

DIED DECEMBER 5TH, 1920.

Isaac Wendell Hubbard, the son of Mark C. and Marian (Wendell) Hubbard, was born on October 24th, 1872, at Greensboro, N. C. He was educated at the public schools of Philadelphia, Pa., Pennington Seminary, and by private tutors.

In 1890, he entered the service of the City of Philadelphia, in the Bureau of Surveys, where he remained for ten years under the late Samuel L. Smedley, M. Am. Soc. C. E., and his successor, George S. Webster, President, Am. Soc. C. E., Chief Engineer. During this time he was engaged on property surveys, on work in connection with the piers and abutments of the Philadelphia and Reading Elevated Railroad, and on the Pennsylvania Avenue Subway and Tunnel. On the completion of the Pennsylvania Avenue Subway, Mr. Hubbard was assigned to the work of making hydrographic surveys of the Delaware and Schuylkill Rivers, removing Schooner Ledge Rock, and other work in connection with port and harbor improvement.

In 1900 he became a partner of Mr. J. Orie Clarke, under the firm name of Clarke and Hubbard. The firm conducted a general engineering practice for

* Memoir prepared by P. Berg, Esq., Philadelphia, Pa.

three years, when the partnership was dissolved and Mr. Hubbard became for a time Engineer of Construction for Ryan and Kelley on the Low Grade Freight Line of the Pennsylvania Railroad at Herrville, Pa. This was a heavy piece of work, involving 80-ft. embankments and masonry bridge construction, and Mr. Hubbard had about 800 men under his direction. On the completion of this contract, in the early part of 1904, he became a partner of Marshall R. Pugh, M. Am. Soc. C. E., under the firm name of Pugh and Hubbard, Civil and Sanitary Engineers, the partnership continuing until the outbreak of the World War in 1917. At that time the senior partner went to France as Major of Engineers, and Mr. Hubbard, after a short period as Superintendent of the Emmons Coal Mining Company, entered the service of the Government as Senior Engineer, Division of Shipyard Plants, U. S. Shipping Board Emergency Fleet Corporation.

During the period from 1900 to 1917, Mr. Hubbard was engaged on the surveys, location, and supervision of roads at Valley Forge Park, Pennsylvania, for the Valley Forge Park Commission; factory building (slow-burning construction) for the H. O. Wilbur Company; surveys, plans, and supervision of a Licorice Manufacturing Plant, at Camden, N. J., which included dredging, bulkheads, wharves, and factory buildings; topographical surveys and development of extensive tracts of land on Long Island and elsewhere; and engineering and location of a great number of interurban trolleys in New Jersey, Pennsylvania, and Ohio.

Becoming interested in contracting and, subsequently, in quarrying, he had charge of a large number of important pieces of work. He was President of the Main Line Stone Company, which operated several quarries and engaged in a general contracting business. Real estate development, construction of roads, concrete, water-bound, and bituminous macadam highways, trolley lines, water-works and sewers, were among his activities.

Mr. Hubbard's duties with the U. S. Shipping Board Emergency Fleet Corporation were administrative, and covered the supervision of plant improvements financed by the Fleet Corporation; review of proposed plant extensions; the amount of money to be expended on such improvements; the policies to be followed; and, finally, the allocation of concrete shipyards.

Leaving the employ of the Government in April, 1920, he became Engineer for the Union Petroleum Company, having charge of the plant construction at Clarendon, Pa., involving an expenditure of \$500 000, and at Marcus Hook, where a lubricating plant was erected, consisting of concrete buildings, 525 ft. of bulkhead, and a large amount of tankage, involving an expenditure of \$1 000 000. He organized the force and had entire charge of the work at both these places, and also at a plant for the same company at New Orleans, La., costing \$500 000.

Mr. Hubbard was a man of tremendous energy, and whatever he undertook he pushed with the greatest vigor. He was a member of St. Paul's Presbyterian Church, West Philadelphia, Pa., and was very active in church affairs. He was also a member of Conrad B. Day Lodge No. 645, F. and A. M., and of St. John's Chapter No. 232, Royal Arch Masons, and of the Engineers' Club of Philadelphia.

On October 5th, 1899, Mr. Hubbard was married to Miss Cecelia McCorkell, who survives him. He also leaves two children, a son and a daughter.

Mr. Hubbard was elected an Associate Member of the American Society of Civil Engineers, on March 1st, 1905, and a Member on January 4th, 1910.

EUGENE WILLETT VAN COURT LUCAS, M. Am. Soc. C. E.*

Died March 8th, 1921.

Eugene Willett Van Court Lucas was born in Mount Vernon, N. Y., on December 21st, 1864. He was appointed to the United States Military Academy in 1883 and was graduated therefrom in 1887. Lieut. Lucas was first assigned to the Artillery, but, in 1888, was transferred to the Engineer Corps of the Army.

After spending two years (1888-90), at the Engineer School of Application, at Willets Point, N. Y., he was detailed as Assistant Instructor in Practical Military Engineering at the U. S. Military Academy until 1892. From 1892 to 1898, he had charge of various engineering works having to do with rivers and harbors and fortifications in North and South Carolina and at Willets Point, N. Y. As Major, U. S. Volunteers, during the war with Spain, he served as Chief Engineer of the 2d Division, 4th Army Corps, in the camps at Tampa and Fernandina, Fla.; as Chief Engineer of the 3d Division, 2d Army Corps, in the camp at Athens, Ga.; and as Chief Engineer of the 2d Army, at Greenville, S. C. Maj. Lucas was honorably discharged from the Volunteer Service on March 2d, 1899.

On his return to his work with the Corps of Engineers, U. S. Army, he was detailed in charge of the North Carolina Fortification and River and Harbor District, with headquarters at Wilmington, N. C., and on the Mississippi River improvements until January 1st, 1906, when he resigned from the service.

After his resignation from the Army, Maj. Lucas became interested in hydraulic projects in the South. Early in 1913, in connection with the late Col. Thomas W. Symonds, U. S. A., Retired, he was appointed by Governor Sulzer as Consulting Engineer for the State of New York on the construction of the State Barge Canal, and continued on this work until 1915. In the turmoil that developed in the State and continued through the term of office of Governor Sulzer, which term ended in his impeachment and removal from office, Col. Lucas performed the duties of his position with marked engineering ability, entirely devoid of the slightest political bias; under the circumstances this was no easy task and will ever remain to his credit as showing that, in the conduct of his work, he operated solely from the point of view of an engineer responsible to the public, as opposed to the exploitation of the State for personal gains.

In 1912, he was appointed Chief of Engineers of thc New York State National Guard, and, later, was assigned to the command of the Twenty-

* Memoir prepared by John A. Bensel, Past-President, Am. Soc. C. E.

second Engineers. In 1916, Col. Lucas went to the Mexican border with this regiment and following its return to New York he retired from the State service.

In 1917, when the United States entered the World War, Col. Lucas again entered the service—a volunteer—as Lieutenant Colonel of the 304th Engineers. He was commissioned as Colonel of the 66th Engineers and assigned to command Camp Laurel, Maryland, in 1918. He was relieved from this duty and ordered to a United States Hospital where he remained until the cessation of hostilities.

Col. Lucas did not resume the active practice of his profession after the World War; he had become interested in some projects in the Southern States which ended rather disastrously, so far as his personal interests were concerned, and cast a cloud over the last year of his life. In this trouble he had the sympathy of those who were permitted within a very reserved exterior. His life was one of accomplishment, both as a military and as a civil engineer, and in his death, which occurred on March 8th, 1921, his professional friends will miss one who throughout his life acted with a high conception of his duty as an engineer and a gentleman.

Col. Lucas is survived by two sons, E. W. Van C. Lucas, Jr., and John D. Lucas. His wife, who was Miss Agnes Daniel, of Wilmington, N. C., died on January 29th, 1916.

Col. Lucas was elected a Member of the American Society of Civil Engineers on April 3d, 1895.

WILLIAM LUDLOW, M. Am. Soc. C. E.*

Died August 30th, 1901.

William Ludlow, the second son of William Handy Ludlow and Frances Louisa (Nicoll) Ludlow, was born at Riverside, Islip, Long Island, N. Y., on November 27th, 1843. He came from distinguished ancestry, having been a direct descendant of Roger Ludlow who was appointed by Oliver Cromwell as Lieutenant Governor, in turn, of Massachusetts and Connecticut, and the first of his family to settle in America. His great-grandfather was an Aide on General Washington's staff, and his father served during the Civil War, having been mustered out with the brevets of Brigadier General and Major General, and subsequently was Speaker of the Assembly of the State of New York. His mother was descended from William Nicoll, who settled at Islip on land granted by Charles II in 1683, and who was the first Royal Secretary of the Colony after its transfer by the Dutch.

William Ludlow received his early education at home, having been tutored by the Rev. Henry M. Davis, the rector of St. John's Protestant Episcopal Church at Islip. In 1853, he was sent to Burlington Academy, Burlington, N. J., and, later, to the University of the City of New York, where he was the recipient of the scholarship presented to the family in recognition of

* Memoir compiled from information supplied by Maj. Gen. William M. Black, U. S. A. (*Retired*), M. Am. Soc. C. E., and on file at the Headquarters of the Society.

the services of his grandfather, Ezra Ludlow, Architect of the University building. In 1860, he entered the United States Military Academy, at West Point, N. Y., from which he was graduated, eighth in his class, on June 13th, 1864.

In 1864, the Federal Government was doing its utmost to bring the Civil War to a close, and the members of the graduating class at West Point were immediately commissioned and sent to the front. Cadet Ludlow received the rank of First Lieutenant, Corps of Engineers, and after a short stay in Washington, D. C., was ordered to report to the Chief Engineer of the Department of Mississippi. He was assigned to the Army of the Cumberland, in which he served as Chief Engineer of the Twentieth Corps until September, 1864. In this capacity, he was engaged in the construction of bridges, the selection of offensive and defensive positions, the design and construction of temporary fortifications, etc. He also took part in the Battle of Peach Tree Creek and was recommended by Gen. Hooker for promotion to brevet rank, "for gallant and meritorious services in laying a bridge across Peach Tree Creek under a severe fire, * * *". He also took part in the battles of the Atlanta Campaign, notably the defense of Allatoona Pass, Ga., on October 5th, 1864, for which he was promoted to the rank of Brevet Captain.

From November, 1864, to March, 1865, Capt. Ludlow served as Chief Engineer of the Army of Georgia on its march to the sea and through the Carolinas. He fought in the battles of Averysborough and Bentonville, the occupation of Goldsborough, and the capture of Raleigh, for which he received commissions as Brevet Major and Brevet Lieutenant Colonel, respectively.

After the close of Sherman's campaign, and a leave of absence, he was ordered, in November, 1865, to Jefferson Barracks, Mo., where he organized the Engineer Depot and the newly authorized Company E of the Battalion of Engineers. He remained in command of the Depot and troops until November, 1867, having, in the meantime, received his commission as Captain in the Corps of Engineers. In December, 1867, he was ordered to report to the late Gen. Q. A. Gillmore, U. S. A., M. Am. Soc. C. E. (then Major, Corps of Engineers, U. S. A.) for duty as his Assistant and served under him at Staten Island, New York, and Charleston, S. C., until November, 1872. While Col. Ludlow was detailed on this duty, he caused the steamer *Henry Burden* to be fitted up for pump dredging; this steamer was the first of a class of hydraulic dredges now so successfully used in river and harbor improvement work.

In November, 1872, Col. Ludlow was appointed Chief Engineer of the Department of Dakota, with headquarters at St. Paul, Minn., which appointment he held until May, 1876. In this position his most important work was his explorations and surveys of the Yellowstone River and through Yellowstone Park and the Black Hills. He was assisted in this work by scientists from various universities who volunteered their services, and the reports of their work went far to bring to the attention of the American people the value and resources of these regions. Col. Ludlow recommended that the care of Yellowstone Park be transferred to the War Department and that it be improved by roads and bridges and opened to the public, which recommendations have since been carried out.

In 1876, Col. Ludlow was ordered to Philadelphia, Pa., as Assistant on river and harbor improvements. He remained in this District until August, 1882, having been promoted to the rank of Major, Corps of Engineers, U. S. A., on June 30th, 1882. During the latter part of this detail he was practically in charge of the work there. Among other things, he improved the methods of the work being done and made such a thorough and comprehensive survey for the improvement of all the navigable waterways that when he was ordered elsewhere a strong effort was made by the citizens of Philadelphia to have him retained in the District. Before he left, he was presented with a memorial signed by the municipal authorities and by the heads of the great commercial, maritime, and railroad interests of the city, "to make an enduring record of their high appreciation of the services of Colonel William Ludlow."

Col. Ludlow's next appointment was as Engineer Secretary to the Lighthouse Board, with headquarters at Washington, D. C. He was retained on this detail until March, 1883, when he was granted a special leave of absence (by Act of Congress February 28th, 1883) to accept the position of Chief Engineer of the Philadelphia Water Department until April, 1886. Col. Ludlow reorganized the Department, correcting the existing conditions, and brought it to an efficient working organization. He had also investigated the question of a water supply for the city and was consulted on that matter from time to time until his death.

In April, 1886, Col. Ludlow was appointed Engineer Commissioner of the District of Columbia which position he held until January, 1888. The experience gained by his connection with the municipal works of Philadelphia assisted him greatly in this work which was undertaken and carried through with his characteristic energy. Among the many difficult problems solved during his incumbency was the extension of Massachusetts Avenue to the west across Rock Creek Valley, which was severely criticized at the time, but which has since proven his wisdom.

After a short tour of duty in Philadelphia as Engineer of the Fourth Lighthouse District, Col. Ludlow was sent to Western Michigan in December, 1888, to take charge of the river and harbor improvement in that part of the State, with headquarters at Detroit, which detail he held until November, 1893. He also served during part of this time as Engineer of the Ninth and Eleventh Lighthouse Districts of the Great Lakes, and had charge of the river and harbor improvements on the eastern coast of Michigan and the waters connecting the Great Lakes. While in charge of this work, he prepared and put into service the project for lighting the narrow and difficult channels between Lakes Superior and Huron, in which, until this time, it had been customary to stop navigation at nightfall to the great detriment of the lake commerce. On his relief from this duty, Col. Ludlow was presented with a set of resolutions by the Lake Carriers' Association of Cleveland, Ohio, and the municipal and commercial authorities of Grand Rapids, Mich., in recognition of his services.

From November, 1893, to April, 1896, Col. Ludlow served as Military Attaché to the United States Embassy at London, England, having been promoted to the rank of Lieutenant Colonel, Corps of Engineers, in 1895.

During this detail, he made an inspection tour of the Suez, Corinth, and Kiel Canals, as well as the maritime canals in Holland.

In April, 1895, Col. Ludlow was commissioned as Chairman of the Nicaragua Canal Board on which he had associated with him Mordecai T. Endicott, Past-President, Am. Soc. C. E., and the late Alfred Noble, Past-President Am. Soc. C. E. This Board was appointed to inspect the route of the proposed Nicaragua Canal and report to President Cleveland on the cost thereon before November, 1895. The Board reported adversely on the proposed cost, and in April, 1896, Col. Ludlow, as Chairman, was recalled from London, where he had returned, to testify in regard to his findings before a Congressional Committee. The report was accepted on his testimony and made the basis for future action on questions relative to the Nicaragua Canal.

Col. Ludlow was next assigned to duty as Engineer of the Third Lighthouse District, with headquarters on Staten Island, New York; but, in 1897, he was transferred to New York City in charge of fortification and river and harbor work which included the improvement of the entrance of New York Harbor. He submitted a report, in 1898, advocating the opening of a deep straight channel from the ocean by the removal of the bar of the so-called East Channel. This project was subsequently adopted and carried out, and, as the Ambrose Channel, now forms the main entrance to the Port of New York.

When war with Spain was declared, Col. Ludlow was ordered to duty as Chief Engineer on the Staff of the Major General commanding. On May 4th, 1898, he was appointed Brigadier General of Volunteers and was sent to Tampa, Fla., as Chief Engineer (temporarily) of the United States forces there under the command of Gen. Shafter, in which capacity he organized the engineer equipment of the Fifth Army Corps. He left Tampa with the Santiago Expedition on June 14th, and on his arrival off Santiago, supervised the transfer of the Cuban Army to Siboney and was assigned to command the First Brigade, Second Division, of the Fifth Army Corps, under Gen. Lawton. The First Brigade took part in the attack on El Caney and Gen. Ludlow's services were highly commended in Gen. Lawton's report on this action. The First Brigade was then moved to Santiago and took a prominent part in that campaign, and Gen. Ludlow was again commended in Gen. Lawton's report.

After the surrender of the Spanish forces, Gen. Ludlow returned to Montauk Point, Long Island, with the First Brigade. In September, 1898, he was commissioned Major General of Volunteers and appointed President of the Board of Officers to make regulations for the transport service. In October, he was assigned to the command of the Second Division, First Army Corps, at Columbus, Ga., where he remained until December, when he was appointed Military Governor of Havana, Cuba, by direction of President McKinley. Gen. Ludlow's orders with this appointment included the charge of "all that relates to the collection and disbursement of the revenues of the port and city and its police, sanitation, and general government"; later, however, he was relieved of that part relating to the collection of revenues. His success as Military Governor was made apparent by the maintenance of order, the cleansing of the city, the organization of the municipal government along

new lines, and the reform in the Courts, schools, sanitary conditions, etc. In April, 1899, he was honorably discharged from the Volunteer Service and immediately recommissioned as Brigadier General, Volunteers, under the new law.

In January, 1900, Gen. Ludlow was appointed Brigadier General, United States Army, and in May of that year, the Department of Havana having been discontinued, he was ordered to return to the United States as President of the War College Board, with headquarters at Washington, D. C. In this capacity he inspected the French and German military establishments and reported thereon and also on needed reforms in the American establishment; at the same time, he presented a project for the proposed War College.

After the completion of his work on the War College Board, Gen. Ludlow was ordered to active service in the Philippines. Before his departure he was honored by being placed in command of the regular army troops participating in the parade at the inauguration of President McKinley on March 4th, 1901.

During the hardships of the Santiago Campaign, Gen. Ludlow's health was badly impaired, and thereafter he suffered from frequent attacks of bronchial trouble. On his arrival in the Philippines, this trouble became worse, and in May, 1901, he was ordered home on the surgeon's certificate of disability. He had never regarded his illness as serious, and this order was obeyed most unwillingly on his part. On his return to the United States, he was taken to his daughter's home at Convent Station, N. J., where he died on August 30th, 1901. He was buried from Trinity Church, New York City, on September 3d, 1901, with military honors, and his ashes were laid in the family burial ground at Islip, Long Island.

Gen. Ludlow was married in 1866 to Miss Genevieve Almira Sprigg, of St. Louis, Mo., who, with his daughter and two grandsons, survived him.

In appearance, Gen. Ludlow was an ideal type of soldier, tall, erect, and graceful, with strong clean cut features. Possessed of a genial manner and a lively sense of humor, his conversation was always interesting and witty. He was a most convincing speaker and writer and his reports were always clear, concise, and logical, possessing an interest rarely associated with public documents. Allotted many and varied duties in his thirty-seven years of active service in the Army, his work was always marked by increased endeavor and results, and as he was ready to praise or censure when such was due, his assistants, civil and military, gave him a devotion and loyalty such as few men are able to inspire.

In character, his strongest traits were his uprightness and hatred of deceit, his devotion to duty, pureness of mind, hospitality, and charity. Gen. Ludlow was deeply religious in his later years and carried his standards into his every day life. His ability, energy, and high aims were continually shown in the furtherance of his plans for increasing the efficiency of the service he so loved and honored, which plans ended for him in their preparation. In all things, he was a man and a gentleman.

The stone covering his burial place contains the following inscription which fittingly describes the soldier and the man:

"A soldier who fought the warfare of life with the same courage and bravery he displayed on the field of battle. Fearless and unswerving in what he believed to be right. Brilliant and versatile, as Engineer, Governor and Commander of troops, he achieved notable success. His life was illumined with bright deeds and with a generous humanity that lifted or shared the burden of others. In the supreme hour of trial his splendid courage was unshaken and he died in the fullest belief of the Life Eternal."

He was a Companion of the Military Order of the Loyal Legion of the United States and member of various other associations.

Gen. Ludlow was elected a Member of the American Society of Civil Engineers on July 5th, 1882, and served as a Director in 1890.

MAX EVERHART SMITH, M. Am. Soc. C. E.*

DIED JANUARY 24TH, 1921.

Max Everhart Smith was born in Berlin, Germany, on October 18th, 1848, of German parents, and was named Max Eberhardt Schmidt, the English equivalent having been assumed in 1918, by permission of the Court. He entered the Preparatory Military Academy at Potsdam in 1859, and a higher Military Academy at Berlin in 1863. In 1867, at the age of 19, he was commissioned a Second Lieutenant of the Artillery Guards, the regiment in which his father had also served as an officer. He was graduated from the Academy of Artillery and Engineer Officers in September, 1869, and was honorably retired in March, 1870.

On the outbreak of the Franco-Prussian War in 1870, Mr. Smith was recommissioned, and was in active military service until the fall of Belfort, in April, 1871.

In May, 1871, he came to America, and, from 1871 to 1875, served as Engineer on the United States Government surveys west of the 100th Meridian, under Lieut. George M. Wheeler, Corps of Engineers, U. S. A.; in the Coast and Geodetic Survey, under the late Professor J. E. Hilgard, M. Am. Soc. C. E., and on Government work in Minnesota and Dakota, under the late Maj. F. U. Farquhar, Corps of Engineers, U. S. A., M. Am. Soc. C. E. Mr. Smith became a naturalized American citizen in 1874.

In 1875, he was appointed by the late Capt. James B. Eads, F. Am. Soc. C. E., as Chief Assistant Engineer on the construction of the South Pass Jetties at the mouth of the Mississippi River, and continued on this work until 1879.

From 1879 to 1881, Mr. Smith was employed as Assistant Engineer on the Government works for the improvement of the Mississippi River at Memphis, Tenn., under Maj. Benyuard, Corps of Engineers, U. S. A., and at St. Louis, Mo., under Maj. O. H. Ernst, Corps of Engineers, U. S. A.

In 1881, he was appointed Engineer in charge of location of the Mexican Central Railroad, on the Tampico-San Luis Potosi Division, and became Chief Engineer of the road in 1884, serving until 1889, when he went to Chicago, Ill.

* Memoir prepared by Gustav Lindenthal, M. Am. Soc. C. E.

In 1890, Mr. Smith opened an office in Chicago and engaged in a general consulting practice which he continued until 1894. During this period he conceived the idea of the continuous train, or moving platform, for the transportation of passengers, and constructed a typical section which was operated at the Columbian Exposition in 1893, the Berlin Exposition in 1895, and, later, at the Paris Exposition in 1900. On each occasion it proved so successful and attracted so much favorable comment both from engineers and the general public that Mr. Smith devoted the greater part of the remainder of his life to its introduction for public use. In 1904, he became President and Chief Engineer of the Continuous Transit Securities Company.

His designs and plans underwent various changes and improvements from time to time, to meet varying conditions, but the general scheme remained the same throughout his entire work. In 1906, he was awarded the John Scott Legacy Medal and Premium by the Franklin Institute of Philadelphia, Pa.

As a result of his activities, Mr. Smith had the satisfaction of seeing his project gain increasing favor and recognition among engineers, and several routes were laid out for its introduction into the transit system of New York City. As he left it, the platform was regarded by transit specialists as the correct solution of the problem of economical and rapid transportation of passengers in congested districts.

Mr. Smith had contributed a number of papers and discussions to the *Transactions* of the Society, notably his paper on "The South Pass Jetties,"* which gained for him the award of the Norman Book Prize by the Society in 1879.

In 1875, he was married to Miss Mary Everhart, of American Colonial and Revolutionary descent, a niece of the late George Plumer Smith, of Philadelphia, Pa., and a great-granddaughter of the Hon. George Plumer, and of Col. Alexander Lowrey, of Pennsylvania.

His death on January 24th, 1921, was a grievous shock to all who knew him. His eldest son, Eads Everhart Smith, died two years before his father. His widow and one son, George Plumer Smith, senior member of the firm Smith and Gallatin, survive him.

Personally, Mr. Smith was a man of extreme refinement and of musical taste and ability. He had composed a number of marches, the last having been a military march entitled "Prepared", which was published by Schirmer and dedicated to the Engineer's Training Battalion of New York. He was a genial companion, an engineer of far vision, a man of unusually broad acquaintance, and left many friends.

This memorial of an Engineer who had been in the front rank of progress and professional attainments, of a man of exceptional culture and most sympathetic and warm friendships, is only an inadequate expression of their high esteem and mourning for him by those who knew him long and well.

Mr. Smith was elected a Member of the American Society of Civil Engineers on May 7th, 1879.

* *Transactions*, Am. Soc. C. E., Vol. VIII (1878), p. 189.

HARRY ELSTNER TALBOTT, M. Am. Soc. C. E.*

Died January 31st, 1921.

Harry Elstner Talbott, the son of John Litler and Sarah Elstner Talbott, was born in Cincinnati, Ohio, on July 4th, 1860. He attended the common schools of his native city, and, later, was graduated from the Engineering Division of the University of Cincinnati.

Mr. Talbott's early experience was in railroad engineering, first with the Kentucky Central and the Cincinnati, Lebanon, and Northern Railroad Companies. In 1883, he was advanced to the position of Division Engineer of the Northern Pacific Railroad Company. Four years later, he joined the Engineering Staff of the Elgin, Joliet and Eastern Railroad Company, and afterward went with the Chesapeake and Ohio Railroad Company.

Going to Dayton, Ohio, in 1892, Mr. Talbott opened a Contracting Engineer's Office and soon built up a substantial business. In 1899, he met Mr. Francis H. Clergue, of Sault Ste. Marie, and this meeting was the beginning of a long association and of a great series of extensive engineering projects which have made the name of Mr. Talbott suggest the marvelous development of this great section of country. The discovery of the Helen Mine at Michipicoten, Ont., Canada, 100 miles north of the "Soo", with its rich iron deposits, made it necessary to build the Michipicoten Branch of the Algoma Central Railroad. With a force of 1 200 men, against the doubts of nearly all who contemplated the giant task, Mr. Talbott built the 100 miles of road in that wild, rough country on schedule time. The material, men, and supplies had to be taken back to Michipicoten before the close of navigation, and the work of construction was largely done during the severe winter.

Bridges, ore docks, and other important structures were also built, all to the great surprise of many who observed the wonderful achievement with admiration. Enduring the hardships of the winter with his men, living in a shack or "wigwam", as they called it, near the work, Mr. Talbott endeared himself to the workers, who loved "The Chief" for his comradeship and genial nature.

In the years following, he constructed the greater number of the buildings of the Algoma Steel Corporation and many of the power dams in St. Mary's Rapids, afterward adding several large units to the steel plant and erecting other extensive buildings. His skill at designing, for accomplishing great tasks with dispatch and thoroughness, for handling men and problems, placed his name on the roster of the great builders of the Continent.

In 1911, Mr. Talbott became interested in pulp and paper. The Lake Superior Paper Company was organized by him and Mr. George H. Mead, Mr. Talbott being elected to the Presidency. This beginning led to his becoming a great figure in the pulp and paper world. In recent years, he had held large interests in The Spanish River Pulp and Paper Company,

* Memoir prepared by George Bancroft Smith, Esq., Dayton, Ohio.

The Ontario Paper Company, Limited, The Mead Pulp and Paper Company, The Mead Fibre Company, The Peerless Paper Company, and others.

As he progressed in his several lines of endeavor, he became a dominant figure in the business world. As President of the City National Bank and the City Trust and Savings Bank, of Dayton, he assumed and held a place among the financial leaders of Southwestern Ohio.

His versatility did not permit of confinement to a restricted line of activity, and he became equally prominent in the manufacturing field. He was one of the founders of The Dayton Metal Products Company, makers of munitions, and of The Dayton-Wright Airplane Company, which organization built nearly 4 000 airplanes during the World War. These two companies were taken over by the General Motors Corporation, in which Company Col. Talbott became a prominent figure. In these enterprises, he was associated with his son, Harold E., and Mr. Charles F. Kettering, the inventor of the Delco starting, ignition, and lighting system for automobiles and the Delco-Light farm lighting outfit.

In 1913, he achieved a great work following the disastrous flood in Dayton, when he assumed the formidable task of cleaning up the wreckage. In recognition of this work, Governor Cox bestowed on him the honorary title of Colonel.

All who knew Col. Talbott were touched by his charming home life. In June, 1887, he was married to Miss Katherine Houk, the daughter of the Hon. George W. Houk, afterward Congressman from the Third District of Ohio. Nine children were born to them, all of whom are living. They are Harold E. Talbott, Mrs. George Shaw Green, Mrs. A. B. Hilton, Nelson Talbott, Mrs. George H. Mead, Mrs. Thomas Hilliard, Miss Lilah, Miss Katherine and Miss Margaret Talbott. His beautiful estate, "Runnymede", in Oakwood, a suburb of Dayton, has been the scene of many great social events, as well as the place of cheer and blessing to neighbors, friends, neighborhood children, and many who have shared the hospitality of Col. and Mrs. Talbott in a thousand ways.

Outstanding men are a great asset to a people. Those who look straight, think sanely, act nobly, live wholesomely, are the arbiters of their own careers and fortunes and an inspiration to thousands who come within the sphere of their activities. How we delight to meet a real fellow. His handshake into which he puts his soul instead of his shadow, his manly pose, his keen sensibilities, his alert mind, all combine to make us realize his potentialities. Such a man was Harry Elstner Talbott, and when he died on January 31st, 1921, it was the close of a splendid career.

As an engineer, capable of great achievements; as a manufacturer, ranking among the foremost; and as a thinker and planner, whose equal is seldom found, he was known throughout the United States and honored because of his ability and likable personality.

Some men can be told how to do a thing and shown how it may be accomplished. Others can conceive a project, provide the ways and means for its consummation, and bring it to a full fruition. Harry Elstner Talbott was that type. He loved big tasks. He thrived on hard jobs. He delighted to carry to

completion what others looked on as impossible. He made one think of "the poor fellow who did not know the thing could not be done, so he just went ahead and did it." Many substantial monuments of his skill and thoroughness stand in various parts of the United States defying time, the elements, and whatever else may assail them.

His generous benevolences were bestowed in an unostentatious way, and countless instances of his tenderness, his humanity, and his philanthropy are known only to the immediate participants. He was a lover of the hunt and of all manly outdoor sports. He rode well, knew the great woods and the haunts of the deer and the moose, and many trophies tell the story of his steady aim.

His ability to judge men was marked, and he used to good advantage his fund of common sense, which was ever apparent. He was resourceful and a man of keen perception, quick decision, and mature judgment.

Col. Talbott was elected a Member of the American Society of Civil Engineers on June 6th, 1900.

WILLIAM GLYDE WILKINS, M. Am. Soc. C. E.*

Died April 12th, 1921.

William Glyde Wilkins, the son of Alvin and Charlotte Glyde Wilkins, was born in Pittsburgh, Pa., on April 16th, 1854. With his parents, he moved to Detroit, Mich., in 1855, and attended the public schools in the latter city.

In September, 1872, he entered the Rensselaer Polytechnic Institute, Troy, N. Y., but after one year's stay at that institution, he left to enter the employ of the Munsing Iron Company, Lake Superior. In 1875, Mr. Wilkins became connected with the Engineering Department of the Pennsylvania Railroad at Pittsburgh, remaining there until September, 1876, when he returned to Troy to complete his engineering education at Rensselaer, from which he was graduated in 1879 with the degree of Civil Engineer.

Mr. Wilkins immediately entered the service of the United States Government and was assigned to the corps that was making a hydrographic survey of the Mississippi River in the vicinity of Fulton, Tenn. He remained there until June, 1880, when he entered the Engineering Department of the Pennsylvania Railroad as Assistant Engineer of Construction, with headquarters at Philadelphia, Pa. In July, 1887, he left the service of this Company and opened an office in Pittsburgh for private practice. On January 1st, 1890, he associated with himself George S. Davison, M. Am. Soc. C. E., under the firm name of Wilkins and Davison. Upon Mr. Davison's retirement from the firm on January 1st, 1900, the business name of the concern was changed to The W. G. Wilkins Company, Mr. Wilkins associating with himself Wilber Macaulay Judd, M. Am. Soc. C. E., and several of his more prominent employees, and this association continued to the time of his death, on April 12th, 1921.

* Memoir prepared by George S. Davison, M. Am. Soc. C. E.

While with the Pennsylvania Railroad Company Mr. Wilkins laid out and had charge of the construction of the branch line from Phœnixville to Fraser, Pa.; the Charles Street Depot at Baltimore, Md., which has been replaced recently by a new structure; the Duquesne Freight Station at Pittsburgh; and the stone arch bridge of the main line of the Pennsylvania Railroad over the Conemaugh River at Johnstown, Pa.

One of his earliest engagements in private practice was the design of the first steel head-frame for a coal mine shaft known to have been built in the United States. Although his practice covered every branch of civil, mining, and mechanical engineering, his intense interest in coal mining and coke making led him to specialize in that class of work, and it is undoubtedly true that he was interested in the design of more mining and coke plants than any other engineer in America.

In the first fifteen years of his private practice Mr. Wilkins laid out and constructed a great number of industrial railways in the Pittsburgh district and also built a large mileage of electric street railways. His intimate knowledge of the coke business caused his appointment, in 1907, as one of the three Trustees of the Estate of William Thaw, Deceased, Coke Trust, owners and lessors of many thousands of acres of coal lands in the Connellsville coke region. He performed the duties of this office to the time of his death.

For a limited period he filled the office of City Engineer of Allegheny, Pa., before that city became a part of Pittsburgh. At that time the city was engaged on a programme of extensive public improvements, which naturally came under his supervision and design.

In addition to his membership in the Society, Mr. Wilkins was a member of the following technical societies: American Institute of Consulting Engineers, American Institute of Mining and Metallurgical Engineers, American Mining Congress, North of England Institute of Mining and Mechanical Engineers, Coal Mining Institute of America, Academy of Science and Arts of Pittsburgh, and the Engineers' Society of Western Pennsylvania. In the latter Society he had served as a Director in 1890 and 1891, and was President in 1896.

At the time of the formation of the Pittsburgh Flood Commission in 1908, he was appointed a member and served actively on the Engineering Committee. This Committee made an exhaustive study of the water-sheds of the Allegheny and Monongahela Rivers, reporting on a plan for controlling the flood-waters of these streams through the construction of impounding reservoirs. Mr. Wilkins gave freely of his time and talent in the preparation of the plans for the report.

As a member of the Chamber of Commerce of Pittsburgh he was most active in the civic affairs of his home city. He served as Director of this body for six years from 1908.

At the session of the Pennsylvania Legislature in 1911, the City of Pittsburgh was granted a new charter, which abolished the plan of entrusting the legislative work of the city to two large bodies of councilmen and placed it in the hands of nine councilmen elected at large. The first body was appointed by the Governor on June 5th, 1911, and Mr. Wilkins was one of those appointed.

When his commission expired, he was elected by popular vote, and continued in office until December 31st, 1915.

As an office-holder, his words and deeds always went direct to the point at issue. His policy was to serve the public to the best of his ability and without favor to any one. His engineering knowledge and skill brought about many reforms in the methods of public improvement.

Mr. Wilkins possessed one of the largest private libraries in the City of Pittsburgh, and was internationally recognized as being the best informed student on the works of Charles Dickens in the world. He delivered many lectures on Dickens and was a member of the Dickens Fellowship. Through the English firm of Chapman and Hall, Limited, of London, he published "Charles Dickens in America", and at the time of his death there was in the hands of his publishers, the Bibliophile Society of America, a new book written by him on "Dickens in Cartoon and Caricature."

As in everything else, Mr. Wilkins was active in religious affairs, being a member of the North Presbyterian Church, of Pittsburgh, which congregation he served as Trustee for many years. He was married to Sarah Rebecca Simmons at Troy, N. Y., on December 29th, 1880, by whom he is survived.

Mr. Wilkins was elected a Member of the American Society of Civil Engineers on December 4th, 1889, and served as a Director during 1909, 1910 and 1911.

ROBERT STUART ARMSTRONG, Assoc. M. Am. Soc. C. E.*

Died July 15th, 1918.

Robert Stuart Armstrong, the son of Richard and Annie Armstrong, was born on February 20th, 1874, at Hamilton, Ont., Canada, where he received his early education. He was graduated from the Hamilton Collegiate.

In 1890, he was employed by Messrs. Jennie and Mundie, Architects, of Chicago, Ill., and while in this position he took up engineering under the late James D. McKee with whom he went to Dayton, Ohio, in 1893 (with the firm of Williams and Andrews, Architects), and to St. Louis, Mo., in 1895. While in St. Louis, Mr. Armstrong acted as Engineer for the School Board Architect, and also assisted the late Professor J. B. Johnson, M. Am. Soc. C. E., of Washington University, in the preparation of material for his book "Materials of Construction", making many of the drawings and plates for this work. Mr. Armstrong also studied perspective drawing and water colors, in which he became very proficient, and his services in this line of work were frequently employed.

During this period of Mr. Armstrong's life his future career shaped itself, but there evidently was a struggle within him as to whether he would follow his early training and inclination toward Architecture or the Profession of Engineering, in the choice of which he undoubtedly was greatly influenced by his teacher and friend, Mr. McKee, a very far-sighted and progressive engineer.

* Memoir prepared by H. Fougner, Assoc. M. Am. Soc. C. E.

Mr. Armstrong was employed by the Koken Iron Works of St. Louis until the fall of 1898, when he was engaged by the Jackson Iron Works of New York City, and subsequently with Post and McCord until 1900, when the plant of the latter firm was acquired by the American Bridge Company. Shortly after the formation of the latter Company, Mr. Armstrong was transferred to the Trenton Plant, at Trenton, N. J., where he remained until February, 1903, when he was transferred to the New York Office as an Estimator and Designer, and made a specialty of designs of office buildings, theatres, armories, and similar structures.

In July, 1904, he was made Engineer of the Brooklyn Plant of the American Bridge Company, in charge of the Engineering Department. In May, 1915, he became Manager of the Brooklyn Plant, which position he held until September, 1915, when he resigned to become Contracting Manager of Milliken Brothers, and when this plant was purchased by the Downey Shipbuilding Company he remained with that company as Works Manager.

In 1918, Mr. Armstrong became Construction Manager for the Carolina Shipbuilding Company of New York City and Wilmington, N. C. Subsequently, he became Fabricating Manager, and left New York City to take up his permanent abode at Wilmington, N. C., where he met his sudden and untimely death, only one week after his arrival there.

Mr. Armstrong was married on June 28th, 1916, to Miss Elizabeth Egginton, of Brooklyn, N. Y., who survives him.

"Bert" Armstrong, as he was familiarly known to his friends, was highly regarded both as engineer and man. As a young man, his proficiency made his services greatly sought after, and when in later years he was called on to direct the work of other men his ability to convey his efficiency to others gained for him the high respect and confidence of his employers.

His sunny nature and sterling qualities made for him a host of friends, and the affectionate friendship of his employers and of the many whose work he was called on to direct is a splendid tribute to his memory.

Mr. Armstrong was elected an Associate Member of the American Society of Civil Engineers on February 6th, 1907.

PAUL JONES BEAN, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 25TH, 1919.

Paul Jones Bean was born in Woodville, Tex., on March 17th, 1884. He received his preparatory education in the common schools of Texas, and was appointed to the U. S. Naval Academy, as Midshipman, on September 2d, 1902. On his graduation, in 1906, he was selected for the Corps of Civil Engineers, U. S. Navy, and was commissioned Assistant Civil Engineer, with the rank of Lieutenant (Junior Grade) in that Corps on April 27th, 1906. He was ordered to the Rensselaer Polytechnic Institute, at Troy, N. Y., for professional instruction, and was graduated from that institution in 1908 with degree of Civil Engineer.

* Memoir prepared by Leonard M. Cox, Commander, C. E. C., U. S. N., M. Am. Soc. C. E.

Immediately following his graduation from Rensselaer, Lieut. Bean was assigned to duty as Inspector for electrical machinery at the works of the General Electrical Company, at Schenectady, N. Y., and during this duty completed the regular Apprentice Instruction Course of that Company. On April 9th, 1910, he was assigned to duty as Assistant to the Public Works Officer, at Norfolk, Va., and, in August, 1911, having completed the necessary examination, he was transferred to the grade of Civil Engineer, with the rank of Lieutenant. In August, 1913, he was detached from the Navy Yard at Norfolk, and ordered to the Naval Station, at Honolulu, Hawaii, as Assistant to the Public Works Officer at that station. During the period of this duty, Mr. Bean contracted tuberculosis, and in November, 1914, was ordered to the Naval Hospital, at Mare Island, Cal., for treatment and observation. After six months' leave, he was sent to the Naval Hospital at Las Animas, Colo., and on March 16th, 1916, was transferred to the retired list.

After his retirement, and treatment in various places, Mr. Bean's health improved to such an extent that he was able again to engage in active work, and he accepted the position of Secretary and Treasurer of the Laurel River Logging Company. It was while on an inspection of this Company's operations at Runion, N. C., that he contracted the disease which ultimately resulted in his death at the Mission Hospital, in Asheville, N. C., on January 25th, 1919.

Mr. Bean was married on February 19th, 1908, to Miss Ethel M. Phillips, of Troy, N. Y., who, with four children, Ethel, Paul, Virginia, and Fields, survives him.

By the death of Mr. Bean, the Corps of Civil Engineers of the United States Navy and the Civil Engineering Profession generally have lost a young man of exceptional promise. He had a genius for organization, and in the comparatively short period of his active service, he had acquired a reputation as an expert in engineering office methods as applied to costs and cost analyses. He was a man of sterling character and was genuinely loved by his subordinates as well as his superiors.

At the time of his death he was a member of the Pafraets Dael Club, of Troy, N. Y., the Mohawk Country Club, of Schenectady, N. Y., and the Army and Navy Club, of Washington, D. C.

Mr. Bean was elected a Junior of the American Society of Civil Engineers on May 2d, 1911, and an Associate Member on December 31st, 1913.

FREDERICK WALLIS DAGGETT, Assoc. M. Am. Soc. C. E.*

Died May 10th, 1921.

Frederick Wallis Daggett, the son of Nellie I. Daggett and the late William H. Daggett, was born in Boston, Mass., on July 26th, 1877. He spent his boyhood days in the place of his birth and received his early education in the public schools of Boston. In 1895, he entered the Lawrence Scientific School, Harvard University, and was graduated in the Class of 1899.

* Memoir prepared by N. A. K. Bugbee, Assoc. M. Am. Soc. C. E.

After leaving college, Mr. Daggett's first engagement was with the Fore River Shipbuilding Company of Quincy, Mass., where he was employed as Draftsman for two years. He then became associated with the United Coke and Gas Company of New York City, as Designer and Resident Engineer on the erection of by-product coke-oven plants and gasworks, with which Company he remained for four years. Subsequently, for several years he was employed, respectively, by the Ransome Concrete Machinery Company, the Ransome and Smith Company, and E. L. Phillips and Company, all of New York City, designing and superintending the construction of reinforced concrete buildings.

In September, 1907, Mr. Daggett began work as Superintendent of Construction of a section of the Catskill Aqueduct, near Peekskill, N. Y., for the Thomas McNally Company, and, later, its successor, John J. Hart. From October, 1909, to April, 1912, he was associated with Fred T. Ley and Company, of Springfield, Mass., as Engineer on the construction and design of filtration plants, during which time he had charge of the construction of a 3 000 000-gal. mechanical filter for the City of Burlington, N. J., and a 5 500 000-gal. plant for the Ayer Mills of the American Woolen Company, at Lawrence, Mass.

During 1912 and the early part of 1913, Mr. Daggett was employed by the Merrill-Ruckgaber Company of New York City, as Engineer in charge of the construction of a 6 000 000-gal. filter plant for the City of Cumberland, Md., and on the completion of this work, he was engaged by Johnson and Fuller as Resident Engineer on a 30 000 000-gal. filter plant at Trenton, N. J. When this work was completed in 1914, Mr. Daggett was appointed by the City Government as Superintendent of the Plant, which position he held at the time of his death, on May 10th, 1921.

In 1914, he was married to Ernestine Giddings, of Waltham, Mass. As a result of this union, one son—Jerome D. Daggett—was born. In August, 1915, Mrs. Daggett died, and on May 26th, 1919, he was married to Frances B. Glasgow, of Burlington, N. J., who, with his mother and son, survives him.

Mr. Daggett was a profound student and a thorough and efficient engineer. He was a man of high character and possessed personal qualities which earned for him a wide circle of friends.

He was a Charter Member of the Engineers Club of Trenton and served for a number of years as a member of its Board of Governors. He was also a member of the Harvard Club of New York City.

Mr. Daggett was elected an Associate Member of the American Society of Civil Engineers on May 1st, 1907.

JAMES RICHARD DONALD MACKENZIE, Assoc. Am. Soc. C. E.*

Died January 25th, 1921.

James Richard Donald Mackenzie, was born in Blackburn, Lancashire, England, on December 25th, 1880. When he was a small child, his parents

* Memoir prepared by Arthur B. Hitchcock, Assoc. M. Am. Soc. C. E.

took up their residence in Canada, where he received his early education, and, later, moved to Massachusetts where his education was completed.

Mr. Mackenzie's first experience in the business world was in May, 1906, when he went to Oakland, Cal., and became connected with the Watson Roofing Company. He soon became Manager of that Company, and, later, was made a partner, in which capacity he was in direct charge of the design and construction of numerous paving, roofing, and water-proofing jobs, involving the use of various grades of bitumens. Mr. Mackenzie severed his connection with the Watson Roofing Company in November, 1907, and established a business of his own in the same line of work.

In 1916, due to a combination of economic conditions and personal affairs, Mr. Mackenzie closed his business in California and moved to Kansas City, Mo., where he became connected with the Kansas City Branch of the Barrett Company. His knowledge of bituminous materials and their derivatives made him a valuable asset to the Sales Organization of this Company, and the major portion of his time was spent in the promotion of paving materials in the State of Kansas.

Mr. Mackenzie had a wide acquaintance among paving contractors and engineers in Kansas, and his opinions and advice were always reliable. In the years that he had followed water-proofing work, he became an authority on the subject. Having always been a thorough student and an intelligent thinker, his studies had led him deeply into the consideration of the whole subject of water-proofing.

Mr. Mackenzie died on January 25th, 1921, in Tucson, Ariz., from an acute attack of pneumonia. He is survived by his widow and a daughter, Margaret, who reside in Kansas City, Mo.

Mr. Mackenzie was elected an Associate of the American Society of Civil Engineers on June 1st, 1920.

FREDERIC BORRADAILE PRICHETT, Jun. Am. Soc. C. E.*

Died September 6th, 1918.

Frederic Borradaile Prichett was born at Philadelphia, Pa., on November 23d, 1890, the son of William Borradaile and Fannie W. Prichett.

After completing his elementary grammar school work, he entered the William Penn Charter School in Philadelphia, on September 16th, 1904. From the time of his entrance until his graduation in 1909, he was identified at all times with the leading activities of the school, taking an active interest in athletics, and in debating and literary activities. He was Editor in Chief of the *Penn Charter Magazine,* the school publication, President of the A. D. Gray Science Club, Manager of the Cricket Team, a member of the School Debating Team, Vice-President of the Literary Society, and a member of the Mandolin and Glee Clubs, and also of the School Orchestra. He was at all times an able and persuasive speaker and was one of three chosen at large from the school to compete in the prize declamation contest at the annual

* Memoir prepared by W. B. Prichett, Esq., North Wynnefield, Philadelphia, Pa.

entertainment in the spring of 1909. He was one of the Commencement speakers and also Historian of his class.

He was a member of Tau Theta Sigma Fraternity of the Penn Charter School, and, in his senior year, was elected a member of Alpha Delta Tau, an honorary society among preparatory schools, corresponding to Phi Beta Kappa.

In the fall of 1909, he entered the Engineering School of the University of Pennsylvania, and in October of the same year became a member of Phi Delta Theta Fraternity. During his career at the University, "Ted" Prichett, as he was known among his friends, was intimately associated with many activities. He took an active interest in social welfare work from the time of his entrance and, in his Senior year, was elected President of the Young Men's Christian Association of the University. In his Senior year, he was appointed Manager of the University basket-ball team, was Valedictorian of his Class, and served on numerous Class Committees. He was a member of the Friars Senior Society, the Plumb Bob Society, and the Civil Engineering Society of the University, and in June, 1913, was graduated with the degree of B. S. in C. E.

During the summer, while attending the University, Mr. Prichett was interested in the practical phases of his profession, working with the Pennsylvania Railroad Company as Rodman and Surveyor and with Hale and Kilburn as Draftsman.

On his graduation from the University of Pennsylvania, he became associated with the Pennsylvania Railroad Company, in the Maintenance of Way Department, and was employed by that Company until the spring of 1915, when he entered the employ of Gibbs and Hill, Engineers, then engaged in the electrification of the Chestnut Hill Branch of the Pennsylvania Railroad. On May 18th, 1916, he was married to Miss Gertrude Bailey Rhoads, of Philadelphia, Pa. In the fall of 1916, he went to Cincinnati, Ohio, in the employ of the American Bridge Company, remaining there until the outbreak of the World War.

In June, 1917, Mr. Prichett returned from Cincinnati and enlisted as a Private in the First City Troop of Philadelphia, and in August went South with that organization to Camp Hancock, Augusta, Ga. On January 5th, 1918, he was assigned to the Third Officers' Training Camp, 28th Division, Camp Hancock, Georgia, and on April 19th, was graduated as one of twenty-six officers from the Division commissioned in the Field Artillery. He was immediately assigned to Headquarters, 109th Field Artillery Regiment. On May 18th, Lieut. Prichett sailed overseas with that regiment arriving at Liverpool, England, on May 31st, 1918. On June 10th he entered on his final course of artillery training at Camp Meucon, France, and was assigned to Battery A, 109th Field Artillery. On August 6th his regiment entrained for the front and from August 11th until his death on September 6th, 1918, he served with distinction as Junior Officer in his battery, on many occasions receiving unstinted commendation from his superior officers for technique in firing his battery and his coolness under enemy fire.

On September 4th, 1918, the 28th Division, as a unit of which his battery was serving, was ordered to cross the Vesle River in the vicinity of Fismes. His battery was ordered to advance in the face of direct enemy observation. On the morning of September 5th his battery was subjected to a terrific barrage. Refusing to seek cover until all his men had first received protection, Lieut. Prichett was wounded by shell fire, from the effects of which he died the following day, September 6th, 1918. He was buried at Coincy, France, but his body subsequently was removed to the now famous National Cemetery at Belleau Woods, near Chateau Thierry.

Lieut. Prichett was elected a Junior of the American Society of Civil Engineers on May 15th, 1917.

Vol. XLVII. SEPTEMBER, 1921. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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PLATES

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Vol. XLVII. SEPTEMBER, 1921. No. 7.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE FLOOD OF JUNE, 1921, IN THE ARKANSAS RIVER, AT PUEBLO, COLORADO

By James Munn* and J. L. Savage,* Members, Am. Soc. C. E.

To be Presented October 5th, 1921

Synopsis

This paper describes the causes and effects of the flood of June, 1921, in the Arkansas River, at Pueblo, Colo., and discusses general plans and estimates for future flood-control works.

A history of former floods is followed by a description of the recent flood, including a discussion of the causes, the resulting property damage, the estimated peak flow and flood volume, the drainage area and rainfall data, and a presentation of alternative plans and estimates for flood-control works.

History of Former Floods

The first flood in the Arkansas Valley known to white settlers occurred in 1864. At that time, Pueblo was little more than a trading post, and the damage was slight. The next flood of unusual volume occurred in 1894. At the time of this flood Pueblo had little or no river protection, and the Arkansas River meandered through the city, cutting its banks and changing its course. This flood did considerable damage by covering the railroad yards and flooding the city to Third and Fourth Streets. After the flood of 1894, the river channel was straightened and substantial levees were built, leaving the river in the condition obtaining at the time of the flood of June, 1921.

With the exception of a flood in the Purgatoire River, a tributary of the Arkansas, in 1908, which washed out the Fort Bent Canal diversion dam and the Amity Canal diversion dam, there has been little damage to irrigation or other works through floods on the Arkansas River or its tributaries in the past twenty years. The minor damages which have occurred from time to time during this period, have been due more to poor construction or insufficient protection than to unusual flood conditions.

* Denver, Colo.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

Description of June, 1921, Flood

On the afternoon of June 2d, 1921, the Arkansas River at Pueblo was carrying 8 100 sec-ft. At 11.30 P. M., the river began to rise rapidly and, at 2.00 A. M., on June 3d, the discharge was about 28 500 sec-ft. At 8.00 A. M., June 3d, the discharge had dropped to 3 500 sec-ft. and, from noon to 5.00 P. M., on June 3d, the discharge was only 2 800 sec-ft. At 5.00 P. M., June 3d, the river started to rise very rapidly, reaching a gauge height of 12.7 ft. and a discharge of 24 000 sec-ft. at 6.40 P. M., where it remained stationary until 7.40 P. M. At 7.40 P. M., it again started to rise rapidly, overtopping the levees and beginning to flood the city at 8.45 P. M., June 3d, with a gauge height of 18.14 ft.

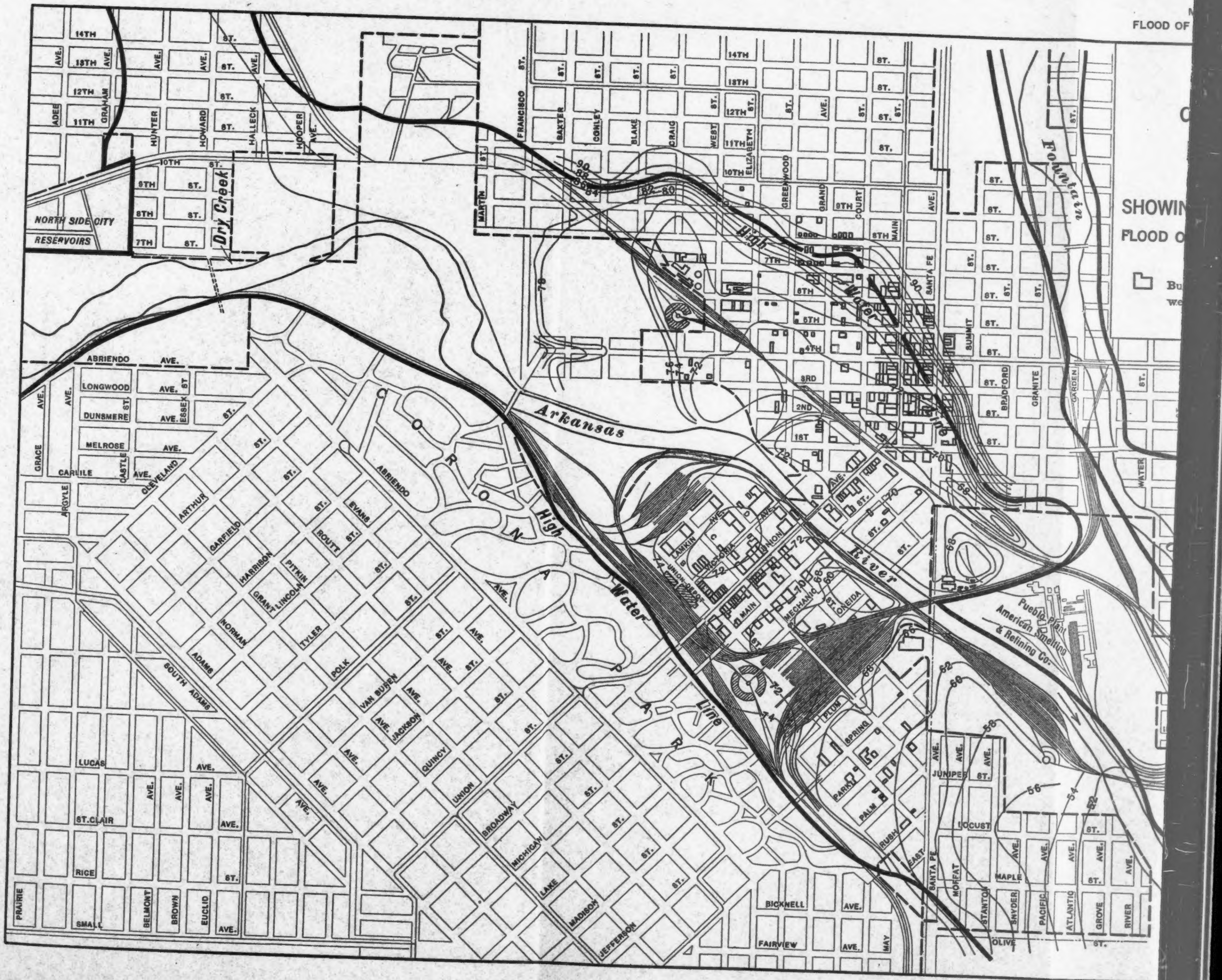
When the river began to overflow its banks, the discharge was probably about 40 000 sec-ft., but from the time of overflow the quantity of water passing through the city cannot be accurately determined, due to the choking of the channel with débris of all kinds. Subsequent levels showed a maximum gauge height of 24.66 ft., and the peak discharge has been roughly estimated at 100 000 sec-ft. The river after overflowing at 8.45 P. M., on June 3d, continued to rise until about 1.30 A. M., of June 4th, when it began to recede. At 4.30 A. M., it had fallen to a gauge height of about 18 ft., with an estimated discharge of about 50 000 sec-ft.

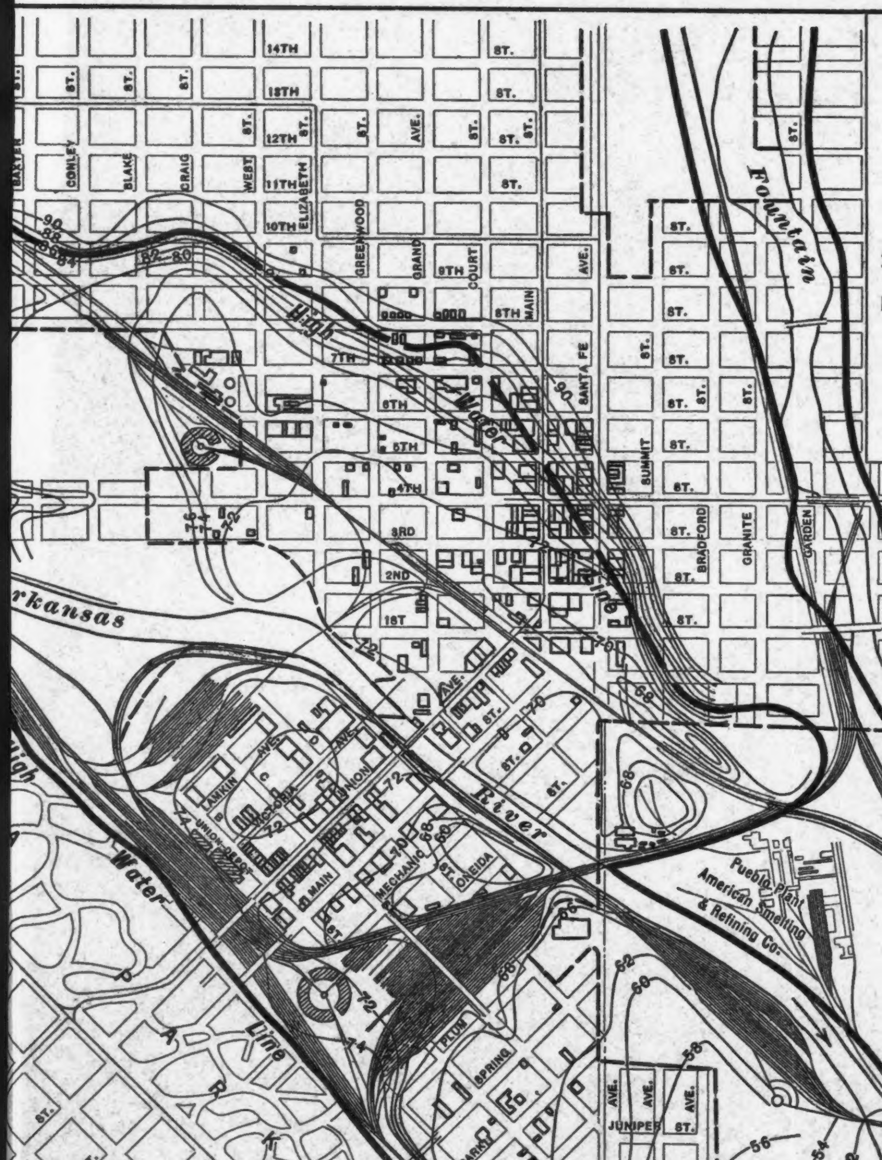
Sometime during the night of June 3d, a flood came down Fountain Creek, a tributary from the north, which joins the Arkansas River at Pueblo. The peak of this flood has been roughly estimated at 50 000 sec-ft. Although this flood receded quickly, it did considerable damage along its own course and added greatly to the damage in the Arkansas Valley below Pueblo.

On Sunday, June 5th, at about 3.00 P. M., another flood in the Arkansas River swept through Pueblo, adding somewhat to the damage and causing renewed alarm. This flood was caused by the destruction of the Schaeffer Dam on Beaver Creek, which released about 3 100 acre-ft. of reservoir storage. Probably no damage would have resulted from this flood if the levees had not already been breached by the greater flood of June 4th. In this connection, it will be noted that the flood of June 5th, resulting from the destruction of the Schaeffer Reservoir, totaled only 3 100 acre-ft., or about one-thirtieth of the whole flood volume.

The flood in the Arkansas River below its junction with Fountain Creek at Pueblo was augmented to a considerable extent by floods in some of the tributaries entering below Pueblo. The St. Charles River added probably 10 000 sec-ft., and this stream did considerable damage along its own course. At La Junta, Colo., the peak in the Arkansas River was probably between 170 000 and 175 000 sec-ft. Below La Junta, the accretions were negligible, and near Lamar, at the Amity Canal diversion dam, the peak flow was estimated at 170 000 sec-ft.

Although the flood peaks, as estimated, were very high, the duration of these high peaks was not long and, consequently, the flood volumes were not as large as might be expected. Rough estimates indicate that a total volume of about 100 000 acre-ft. passed through Pueblo from the Arkansas River and that


FLOOD OF
SHOWIN
FLOOD O
Fountain
Arkansas
River
Dry Creek
High Water Line
CORONA PARK
NORTH SIDE CITY
RESERVOIRS
Pueblo Plant
American Smelting
& Refining Co.
ABRIENDO AVE.
LONGWOOD AVE.
DUNSMERE AVE.
MELROSE AVE.
CARLILE AVE.
SOUTH ADAMS
LUCAS
ST. CLAIR
RICE
SMALL
PRAIRIE
BELMONT
BROWN
EUCLID
BICKNELL AVE.
FAIRVIEW AVE.
UNION DEPOT
SANTA FE
SUMMIT
BRADFORD
GRANITE
GARDEN
WATER
JUNIPER ST.
LOCUST
MAPLE
OLIVE
MOFFAT
STANTON
SNYDER
PACIFIC
ATLANTIC
GROVE
RIVER



Arkansas
River
High
Water
Line
High Water
Fountain
P A R K
Pueblo Plant
American Smelting
& Refining Co.
Union Depot
14TH
13TH
12TH
11TH
10TH
9TH
8TH
7TH
6TH
5TH
4TH
3RD
2ND
1ST
BAXTER
CONLEY
BLAKE
CRAIG
WEST
ELIZABETH
GREENWOOD
GRAND
COURT
MAIN
SANTA FE
SUMMIT
BRADFORD
GRANITE
GARDEN
LAMKIN
VICTORIA
UNION
MECHANIC
ONEIDA
PLUM
SPRING
PARK
PALM
RUSH
EAST
BICKNELL
FAIRVIEW
MAY
LAKE
MADISON
JEFFERSON
JUNIPER
LOCUST
MAPLE
OLIVE
MOFFAT
STANTON
SNYDER
PACIFIC
ATLANTIC
GROVE
AVE.
ST.

PLATE II.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1921.
MUNN AND SAVAGE ON
FLOOD OF JUNE, 1921, AT PUEBLO, COLO.

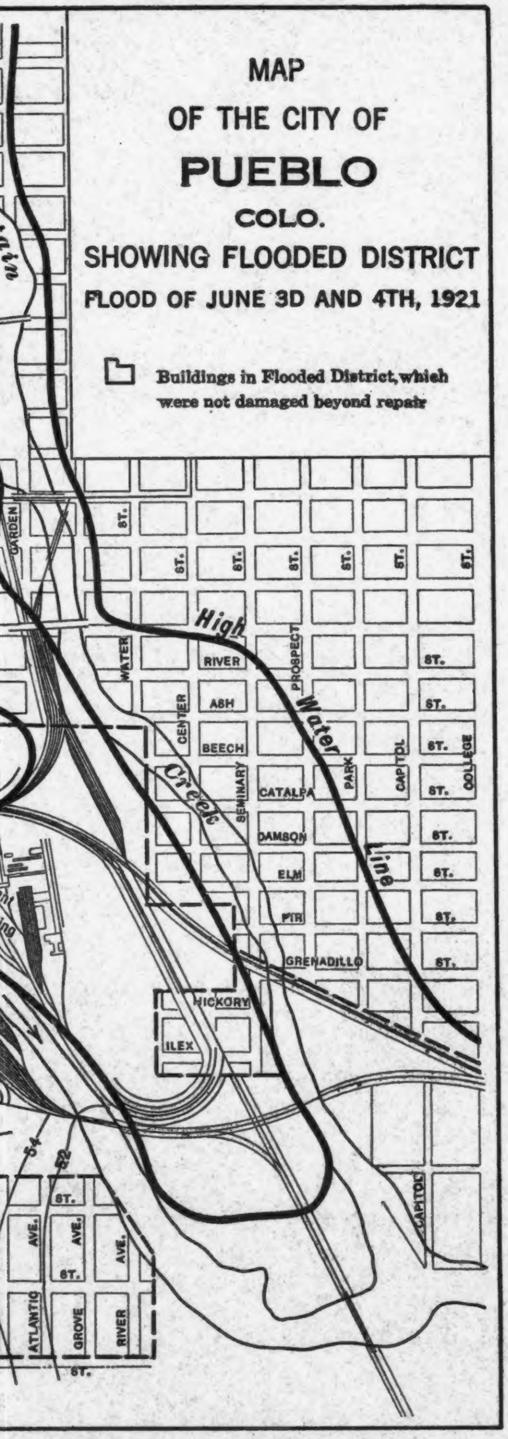
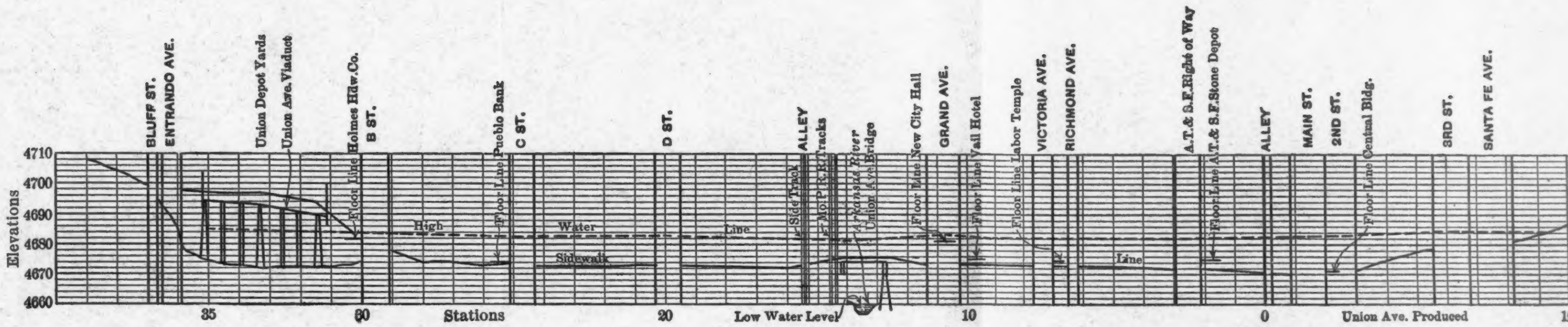
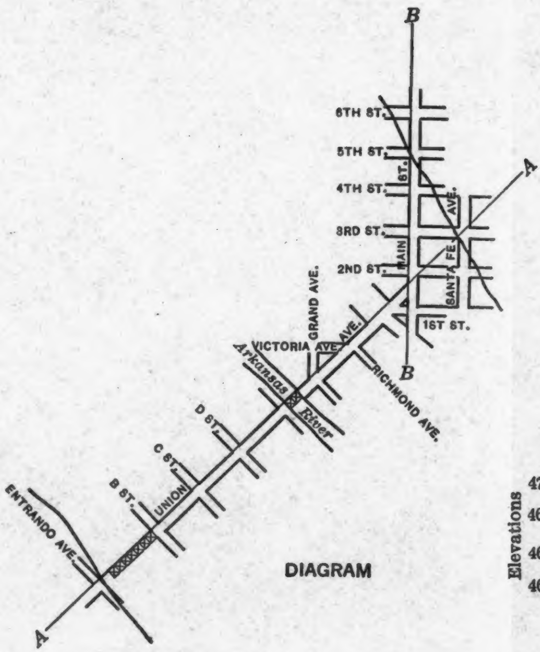


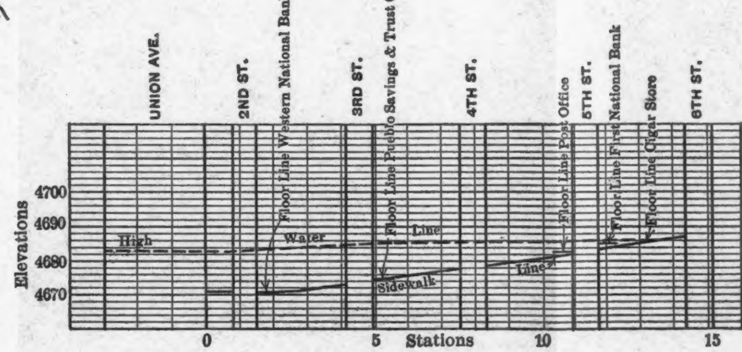
PLATE III.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1921.
MUNN AND SAVAGE ON
FLOOD OF JUNE, 1921, AT PUEBLO, COLO.



UNION AVENUE
(BLUFF STREET TO SANTA FE AVE.)
LINE *A-A*

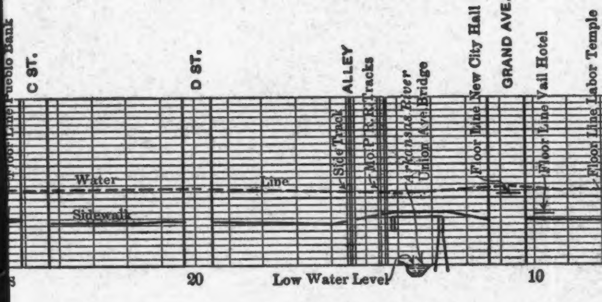


DIAGRAM

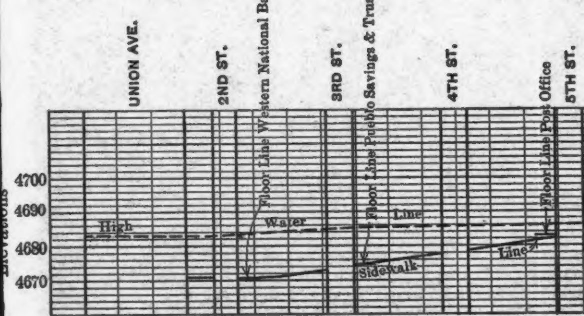


MAIN STREET
(UNION AVE. TO 6TH STREET)
LINE *B-B*

PROFILES
ACROSS FLOODED DISTRICT
CITY OF
PUEBLO
COLORADO
FLOOD OF JUNE 3RD AND 4TH, 1921

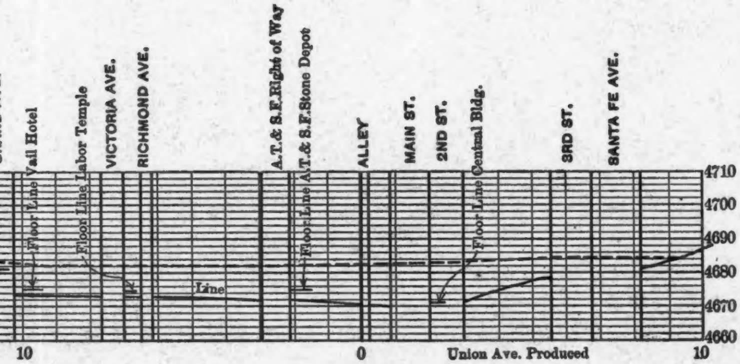


UNION AVENUE
(BLUFF STREET TO SANTA FE AVE.)
LINE *A-A*

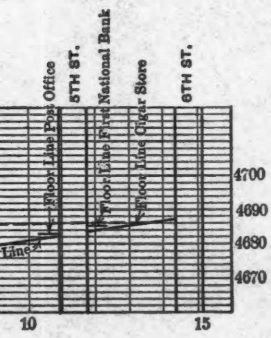


MAIN STREET
(UNION AVE. TO 6TH STREET)
LINE *B-B*

PLATE III.
PAPERS, AM. SOC. C. E.
SEPTEMBER, 1921.
MUNN AND SAVAGE ON
FLOOD OF JUNE, 1921, AT PUEBLO, COLO.



E.)



ET)

PROFILES
ACROSS FLOODED DISTRICT
CITY OF
PUEBLO
COLORADO
FLOOD OF JUNE 3RD AND 4TH, 1921

FIG. 1.—AEROPLANE VIEW OF FLOOD OF JUNE, 1921, IN THE ARKANSAS RIVER AT PUEBLO, COLO.

FIG. 2.—VIEW OF MICKALLS' PACKING PLANT, AND ARKANSAS RIVER, PUEBLO, COLO.

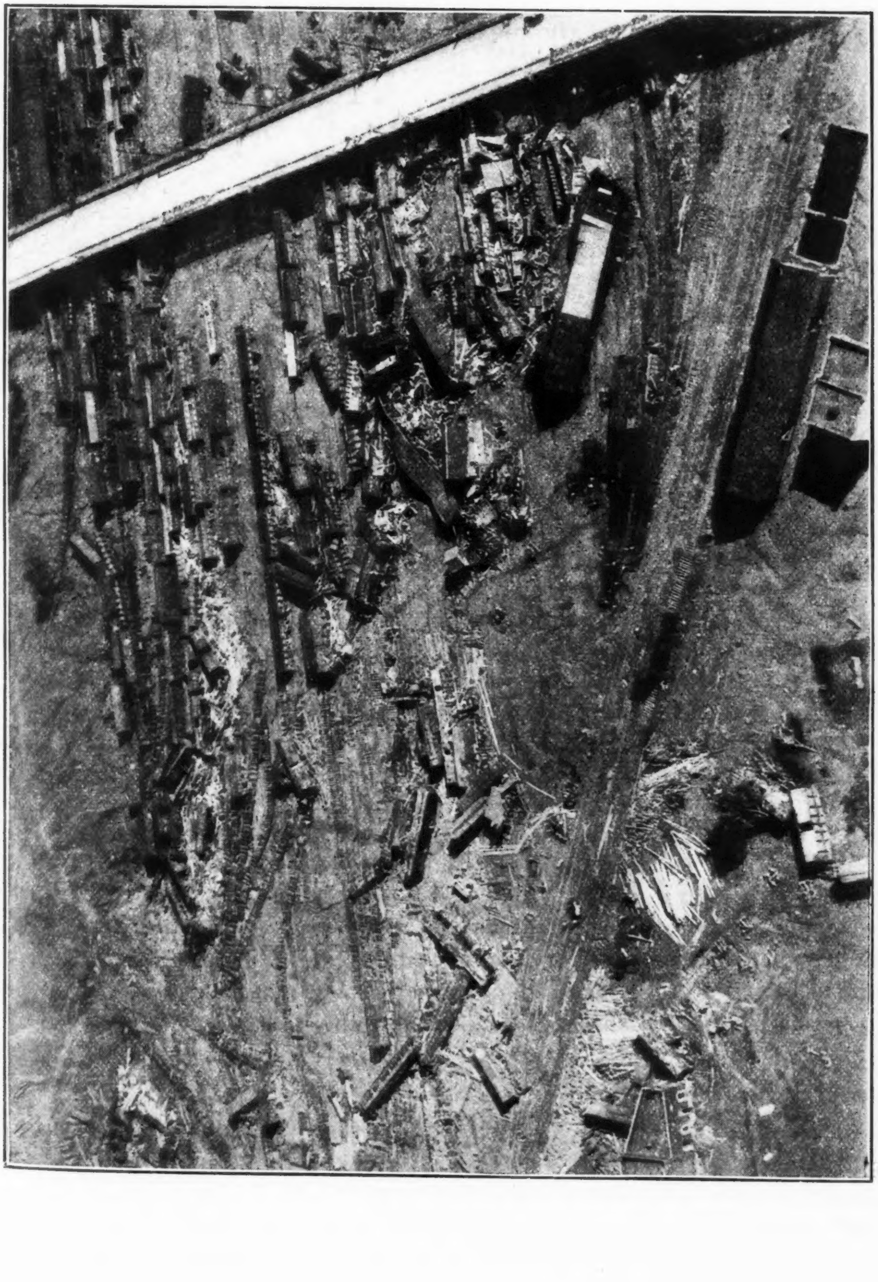


FIG. 3.—AEROPLANE VIEW OF FLOOD DAMAGE IN RAILROAD YARDS, PUEBLO, COLO.

from about 50 000 to 75 000 acre-ft. was added by the Fountain, St. Charles, and other streams down the valley during the period of the flood.

Flood Losses

Preliminary estimates indicate that the property losses will total as follows:

Federal, State, and County	$900 000
Municipal	800 000
Real estate (city and town)	3 420 000
Personal (city and town)	3 575 000
Farm	3 675 000
Irrigation works	1 275 000
Railroads	4 685 000
Public utilities	500 000
Other property not included above	250 000
Total actual damage	$19 080 000

In addition to the actual damages, as estimated in the foregoing, there are intangible losses which are not covered in the estimates. The loss to railroads, public utilities, manufacturers, merchants, and business concerns generally, from the suspension of business, will aggregate a large sum. Likewise, the depreciation of property values, both within and without the flooded districts, if susceptible of estimate, would add materially to the actual losses.

In connection with the depreciation of property values, it should be noted that the ultimate depreciation will depend to a large extent on the scope of the flood-prevention measures finally adopted. If heroic measures are undertaken, and the City of Pueblo and other interests down the valley are made safe against future floods, the ultimate depreciation of property values will be comparatively little. However, if adequate flood protection is not provided, the depreciation of values will undoubtedly be very great.

The City of Pueblo suffered an actual loss of about $10 000 000 out of the total of $19 080 000. The effect of this loss can be realized by comparing it with the city's assessed valuation of $33 000 000. Besides suffering these staggering losses, the city was left with an expensive clean-up job. Fortunately, the War Department undertook extensive sanitary work, including the cleaning away of mud and débris and the repairing of damage to the city water-works system, furnishing the funds and equipment, in addition to a company of engineers, for such work.

Estimated Peak Flow and Volume

Probably the best estimate of the peak flow and the volume of the flood that can be made without extensive field surveys, is contained in a preliminary report to the State Engineer of Colorado, by Mr. R. G. Hosea, Deputy State Engineer. This report contains a table which is reproduced here as Table 1.

TABLE 1.—Quantitative Estimate of Pueblo Flood.

Date. 1921.	Time.	Gauge.	Second-feet.	Average second-feet.	Hours.	Acre-feet.
June 2	5.30 P. M.	7.8	7 900			
				8 100	6	4 050
June 2	11.30 P. M.	8.0	8 300			
				18 400	$2\frac{1}{2}$	3 833
June 3	2.00 A. M.	13.7	28 500			
				16 000	6	8 000
June 3	8.00 A. M.	5.5	3 500			
				3 150	4	1 050
June 3	12.00 M.	5.0	2 800			
				2 800	5	1 160
June 3	5.00 P. M.	5.0	2 800			
				10 350	$1\frac{1}{6}$	1 000
June 3	6.00 P. M.	11.2	17 900			
				20 950	$\frac{1}{2}$	873
June 3	6.40 P. M.	12.7	24 000			
				24 000	1	2 000
June 3	7.40 P. M.	12.7	24 000			
				34 500	1	2 875
June 3	8.30 P. M.	16.85 (Max. channel could carry)	45 000			
June 3	8.45 P. M.	overflow		72 500	5	30 200
June 4	1.30 A. M. (Est. from reports)	Max.	100 000 (Est.)			
				75 000	3	18 750
June 4	4.30 A. M.	18.0	50 000 (Est.)			
				45 000	4	22 500
Total						96 296

The main part of the flood started at 5.00 P. M. on June 3d, and totaled more than 78 000 acre-ft. in the following 18 hours. At 10.30 A. M., June 4th, the river was still flowing 40 000 sec-ft., as estimated by Mr. Hosea. There is, however, no information on which to base an accurate estimate of the volume added after this time. It is probable that fully 20 000 acre-ft. was added and that the main part of the flood, starting at 5.00 P. M., June 3d, and ending some time during the night of June 4th, totaled about 100 000 acre-ft. Rough estimates indicate that the peak flow in Fountain Creek was about 50 000 sec-ft., and that the flood volume was about 50 000 acre-ft. A hydrograph of the flood in the Arkansas River has been plotted corresponding with the data given in Table 1, which is shown on Fig. 8.

Drainage Area and Rainfall Data

The drainage area which contributed to the flood in the Arkansas River through Pueblo, embraces 1 740 sq. miles, between Canon City and Pueblo. The drainage area tributary to Fountain Creek, which creek joins the Arkansas River at Pueblo, includes 930 sq. miles. These drainage areas consist largely of barren land ranging in elevation from 4 600 to 9 000 ft. above sea level. The areas at different elevations are shown by Table 2.

The drainage area on the Arkansas River above Canon City, embracing 3 060 sq. miles, contributed very little to the flood, and this area can be eliminated from consideration in estimating the run-off. Although the rainfall was quite general over the tributary area of 1 740 sq. miles, the excessive rainfall was confined to a comparatively small portion of this area. The total volume of water which passed Pueblo has been estimated at 100 000 acre-ft. and

FIG. 4.—WRECKAGE IN YARDS OF DENVER AND RIO GRANDE RAILWAY COMPANY, ARKANSAS RIVER IN BACKGROUND.

FIG. 5.—WRECKAGE AT RAILROAD CROSSING OVER THE ARKANSAS RIVER.

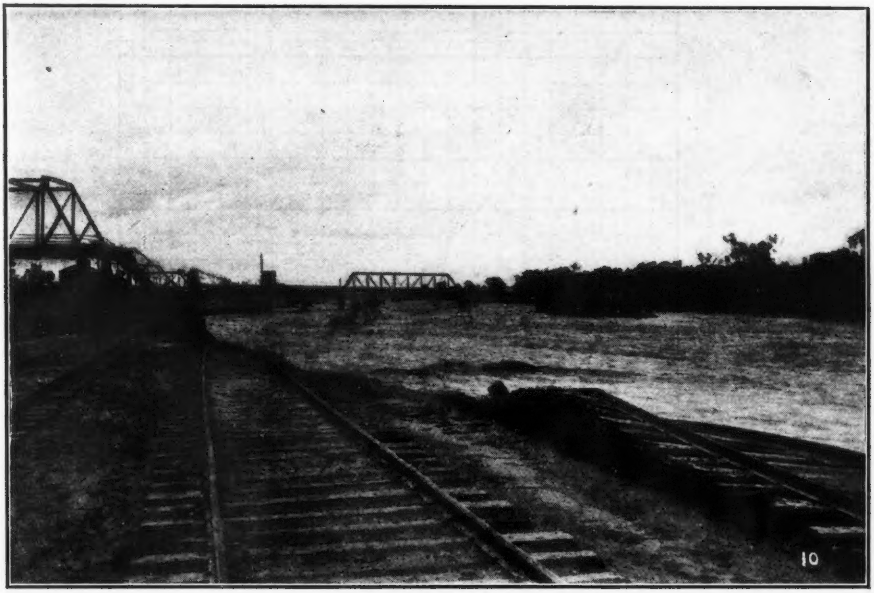


FIG. 6.—BRIDGE AND TRACK DESTRUCTION BY FOUNTAIN CREEK AT FIRST STREET, PUEBLO, COLO.

FIG. 7.—VIEW OF NORTH UNION AVENUE, PUEBLO, COLO., LOOKING TOWARD THE ARKANSAS RIVER.

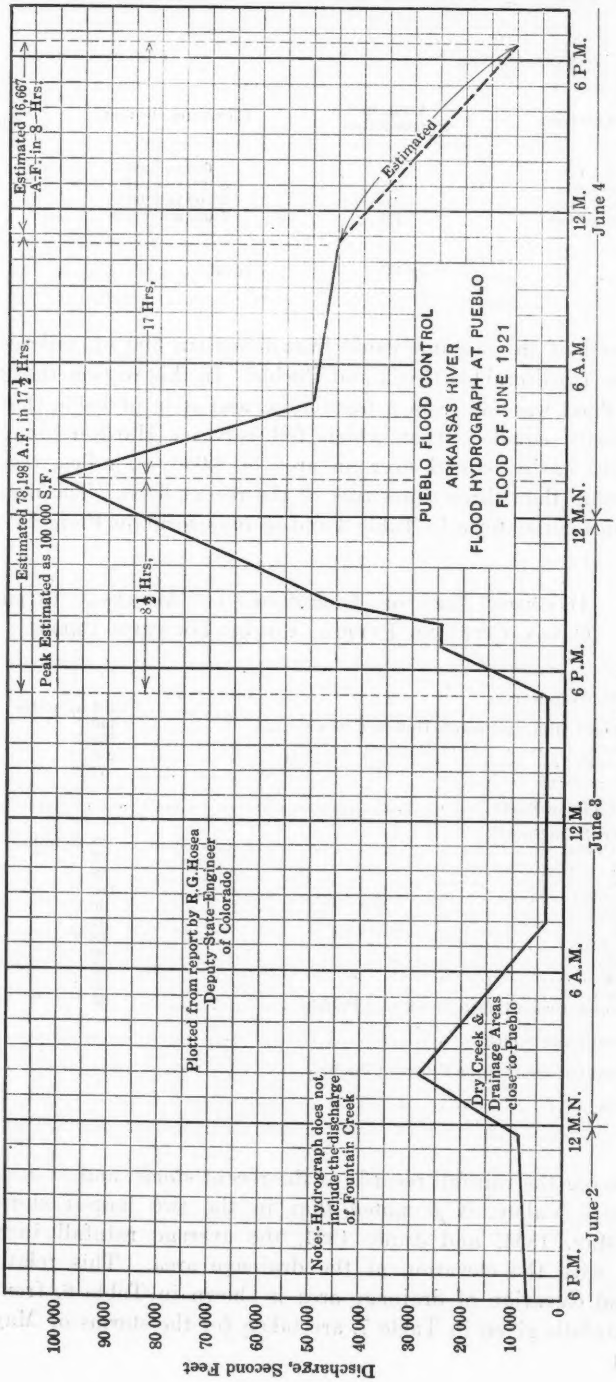

PUEBLO FLOOD CONTROL
ARKANSAS RIVER
FLOOD HYDROGRAPH AT PUEBLO
FLOOD OF JUNE 1921
Estimated 78,198 A.F. in 17½ Hrs.
Peak Estimated as 100 000 S.F.
Estimated 16,667 A.F. in 8 Hrs.
8½ Hrs.
17 Hrs.
Estimated
Plotted from report by R. G. Hosea Deputy State Engineer of Colorado
Note: Hydrograph does not include the discharge of Fountain Creek
Dry Creek & Drainage Areas close to Pueblo
Discharge, Second Feet
100 000
90 000
80 000
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
6 P.M.
12 M.N.
6 A.M.
12 M.
6 P.M.
12 M.N.
6 A.M.
12 M.
6 P.M.
June 2
June 3
June 4

FIG. 8.

TABLE 2.—DRAINAGE AREAS AT DIFFERENT ELEVATIONS.

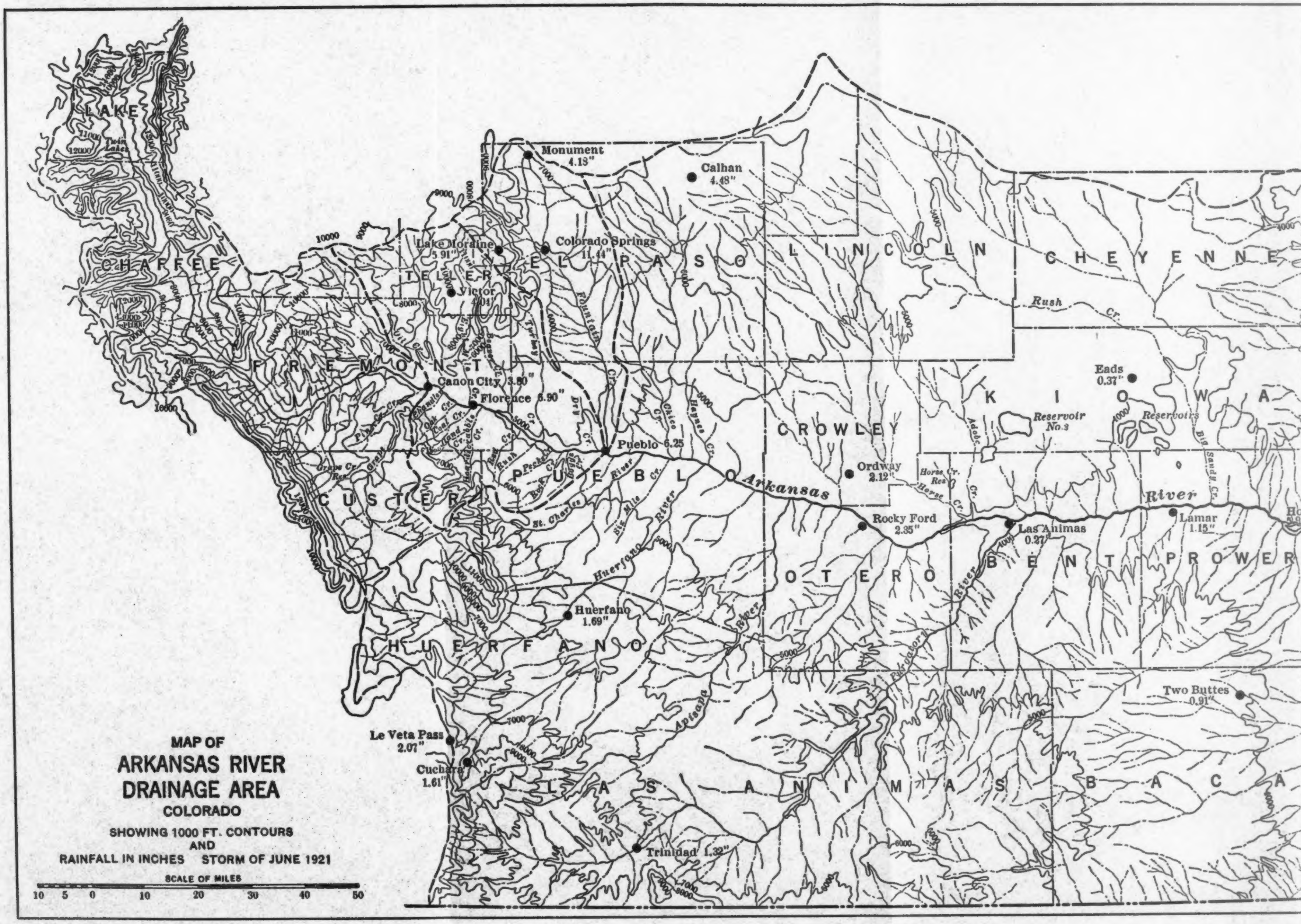
ARKANSAS RIVER.		FOUNTAIN CREEK	
Elevation, in feet.	Area, in square miles.	Elevation, in feet.	Area, in square miles.
4 000 to 5 000	88	4 000 to 5 000	38
5 000 to 6 000	632	5 000 to 6 000	337
6 000 to 7 000	229	6 000 to 7 000	273
7 000 and over	791	7 000 and over	282
Total....................	1 740	Total....................	930

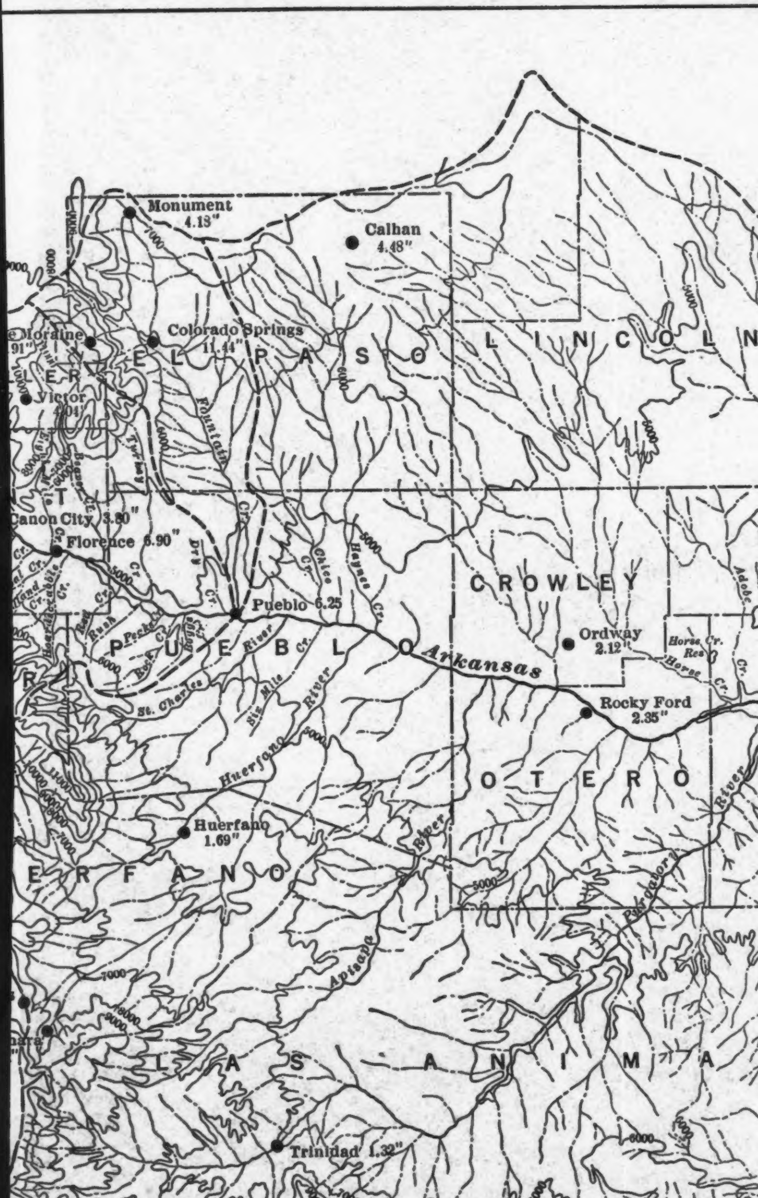
probably one-half this volume came from less than 300 sq. miles of drainage area between Hardscrabble Creek and Pueblo. In this respect the storm which caused the flood was far from a maximum, and it is probable that a rainfall of an intensity equal to that which fell between Hardscrabble Creek and Pueblo might easily extend over an area of 1 000 sq. miles, resulting in a run-off of more than three times that of the recent flood. The drainage areas are shown in tabular form in Table 3 and in map form on Plate IV and Fig. 9.

TABLE 3.—DRAINAGE AREA OF TRIBUTARIES OF ARKANSAS RIVER, BETWEEN CANON CITY AND PUEBLO, AND OF FOUNTAIN CREEK.

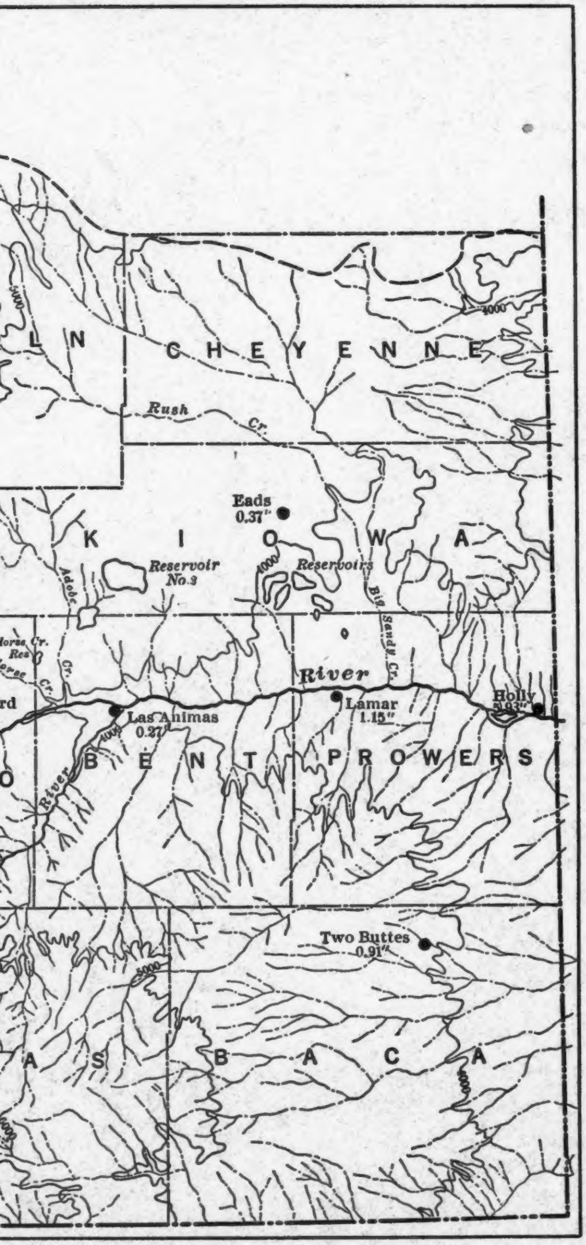
Arkansas River, from North:		
Oil Creek....................	456 sq. miles	
Six-Mile, Eight-Mile, and Birch Hollow Creeks....................	140 " "	
Beaver Creek....................	260 " "	
Turkey Creek....................	215 " "	
Dry Creek....................	71 " "	
Total from North....................		1 142 sq. miles
Arkansas River, from South:		
Chandler Creek....................	38 sq. miles	
Oak Creek....................	72 " "	
Coal Creek....................	28 " "	
Hardscrabble Creek....................	186 " "	
Ritchie Gulch....................	40 " "	
Red Creek....................	42 " "	
Rush Creek....................	34 " "	
Peck Creek....................	46 " "	
Rock Creek....................	67 " "	
Boggs Creek....................	25 " "	
Small creeks between Boggs Creek and Pueblo....................	20 " "	
Total from South....................		598 sq. miles
Total between Canon City and Pueblo....................		1 740 sq. miles
Fountain Creek....................		930 sq. miles

In studying the rainfall records of the recent storm, and of other storms in the Arkansas Valley, it is noted that in the two largest storms, namely, those of May, 1894, and June, 1921, the average rainfall increases quite uniformly with the elevation of the drainage area. This relation between rainfall and elevation of drainage area is shown in Table 6, from which the average rainfalls given in Table 7, are taken for the storms of May, 1894, and June, 1921.


MAP OF
ARKANSAS RIVER
DRAINAGE AREA
COLORADO
SHOWING 1000 FT. CONTOURS
AND
RAINFALL IN INCHES STORM OF JUNE 1921
SCALE OF MILES
10 5 0 10 20 30 40 50
LAKE
CHAFFEE
FREMONT
TELLER
EL PASO
LINCOLN
CHEYENNE
KIOWA
CROWLEY
PUEBLO
CUSTER
OTERO
BENT
PROWERS
HUERFANO
LAS ANIMAS
BACA
Monument 4.13"
Calhan 4.48"
Lake Moraine 5.91"
Colorado Springs 11.44"
Victor
Canon City 3.80"
Florence 5.90"
Pueblo 6.25
Eads 0.37"
Reservoir No.3
Reservoirs
Ordway 2.12"
Rocky Ford 2.35"
Las Animas 0.27"
Lamar 1.15"
Two Buttes 0.91"
Huerfano 1.69"
Le Veta Pass 2.07"
Cuchara 1.61"
Trinidad 1.32"
Arkansas
River
Rush Cr.
Fountain
Huerfano River
St. Charles River
Apishapa
Purgatoire River



Monument
4.13"
Calhan
4.48"
Colorado Springs
11.44"
E L P A S O
L I N C O L N
Victor
4.04"
Canon City 3.80"
Florence 6.90"
Fountain
Pueblo 6.25
C R O W L E Y
Ordway
2.12"
P U E B L O
Arkansas
St. Charles
Huerfano River
Rocky Ford
2.35"
O T E R O
Huerfano
1.69"
E R F A N O
Apishapa
L A S A N I M A
Trinidad 1.32"


CHEYENNE
Rush Cr.
Eads 0.37"
KIOWA
Reservoir No.3
Reservoirs
Big Sandy Cr.
Adobe Cr.
River
Lamar 1.15"
Holly
Las Animas 0.27"
BENT
PROWERS
Two Buttes 0.91"
BACA
4000
5000

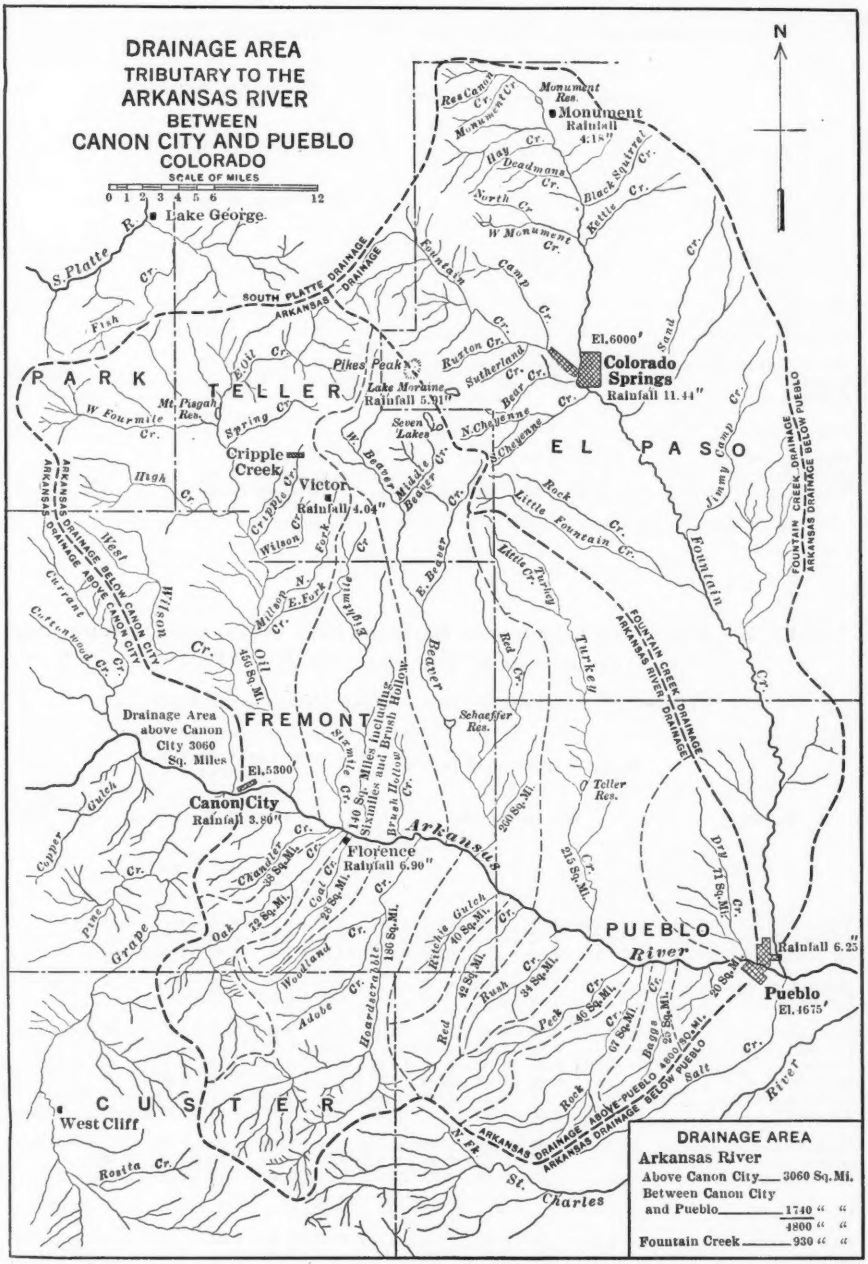

DRAINAGE AREA
TRIBUTARY TO THE
ARKANSAS RIVER
BETWEEN
CANON CITY AND PUEBLO
COLORADO
SCALE OF MILES
0 1 2 3 4 5 6 12
Lake George
Monument
Rainfall 4.18"
Colorado Springs
Rainfall 11.44"
El.6000'
Pikes Peak
Lake Moraine
Rainfall 5.91"
Cripple Creek
Victor
Rainfall 4.04"
PARK
TELLER
EL PASO
FREMONT
PUEBLO
CUSTER
SOUTH PLATTE DRAINAGE
ARKANSAS DRAINAGE
Drainage Area above Canon City 3060 Sq. Miles
El.5300'
Canon City
Rainfall 3.80"
Florence
Rainfall 6.90"
Pueblo
El.4675'
Rainfall 6.23"
West Cliff
ARKANSAS DRAINAGE BELOW CANON CITY
ARKANSAS DRAINAGE ABOVE CANON CITY
FOUNTAIN CREEK DRAINAGE
ARKANSAS RIVER DRAINAGE
ARKANSAS DRAINAGE ABOVE PUEBLO
ARKANSAS DRAINAGE BELOW PUEBLO
DRAINAGE AREA
Arkansas River
Above Canon City 3060 Sq. Mi.
Between Canon City and Pueblo 1740 " "
4800 " "
Fountain Creek 930 " "

FIG. 9.

TABLE 4.—DAILY AND CUMULATED RAINFALL OVER ARKANSAS DRAINAGE AREA IN COLORADO DURING THE STORM OF MAY, 1894.

(Order of Stations is from West Proceeding East)

Station	Daily Rainfall, in Inches. Day of Month.						Cumulated Rainfall, in Inches. Days of Month.					Rainfall to:
	28	29	30	31	1	2	28-29	28-30	28-31	28-1	28-2	
Lake Moraine	......	......	5.50	2.00	......	......	......	5.50	7.50	......	......	6.00 P. M.
Canon City	......	0.75	4.31	......	......	......	......	5.06	......	......	......	....
Husted	0.03	0.08	1.15	0.84	0 22	......	0.11	1.26	2.10	2.32	......	6.00 A M.
Glen Eyrie	......	......	3.13	1.58	0.15	......	......	3.13	4.71	4.86	......	....
Colorado Springs	......	0.08	2.95	1.44	0.50	......	0.08	3.03	4.47	4.97	......	....
Divide Exp. Station	......	0.02	1.65	1.82	......	......	0.02	1.67	3.49	......	......	12.00 P. M.
Hamps	......	0.06	......	2.70	......	......	0.06	0.06	2.76	......	......	7.00 P. M.
Rocky Ford	......	......	......	3.50	......	......	......	......	3.50	......	......	....
Las Animas	......	......	0.07	1.09	......	......	......	0.07	1.16	......	......	7.00 P. M.
Cheyenne Wells	......	......	2.00	......	......	......	......	2.00	......	......	......	....
Springfield	......	4.00	......	0.10	......	......	4.00	4.00	4.10	......	......	....
Vilas	0.14	0.62	0.93	0.76	......	......	0.76	1.69	2.45	......	......	....

TABLE 5.—DAILY AND CUMULATED RAINFALL OVER ARKANSAS DRAINAGE AREA IN COLORADO DURING THE STORM OF JUNE, 1921.

(Order of Stations is from West Proceeding East)

Station	Daily Rainfall, in Inches. Day of Month.						Cumulated Rainfall, in Inches. Days of Month.					Rainfall to:
	2	3	4	5	6	7	2-3	2-4	2-5	2-6	2-7	
Victor	......	0.03	2.08	1.55	0.37	0.01	0.03	2.11	3.66	4.03	4.04	4.00 P. M.
Canon City	......	0.30	2.35	0.75	0.40	......	0.30	2.65	3.40	3.80	......	....
La Veta Pass	......	0.98	0.89	......	0.20	......	0.98	1.87	1.87	2.07	......	....
Lake Moraine	......	0.65	3.68	1.40	0.18	......	0.65	4.33	5.73	5.91	5.91	3.30 P. M.
Florence	......	0.99	3.31	2.47	0.13	......	0.99	4.30	6.77	6.90	......	....
Monument	0.06	......	2.90	0.82	......	0.40	0.06	2.96	3.78	3.78	4.18	4.00 P. M.
Colorado Springs	0.35	5.00	4.40	1.26	0.42	0.01	5.35	9.75	11.01	11.43	11.44	12.00 M.
Pueblo	1.94	1.64	1.45	1.12	0.09	0.01	3.58	5.03	6.15	6.24	6.25	3.25 P. M.
Huerfano	......	......	1.06	0.56	0.04	0.03	......	1.06	1.62	1.66	1.69	6.30 P. M.
Cuchara Camps	......	......	0.86	0.21	0.12	0.42	......	0.86	1.07	1.19	1.61	12.00 M.
Trinidad	......	0.20	0.55	0.30	......	0.27	0.20	0.75	1.05	1.05	1.32	3.15 P. M.
Calhan	......	......	3.26	0.83	0.39	......	......	3.26	4.09	4.48	......	7.30 P. M.
Ordway	0.25	......	0.90	0.75	0.19	0.03	0.25	1.15	1.90	2.09	2.12	....
Rocky Ford	......	......	1.40	0.80	0.15	......	......	1.40	2.20	2.35	......	....
Las Animas	......	0.27	......	......	......	......	0.27	......	......	......	......	8.00 P. M.
Eads	......	0.03	0.13	0.21	......	......	0.03	0.16	0.37	......	......	5.00 P. M.
Lamar	......	......	0.50	......	0.65	......	......	0.50	......	1.15	......	5.00 P. M.
Two Buttes	......	......	0.22	0.15	0.30	0.24	......	0.22	0.37	0.67	0.91	....
Holly	5.88	......	0.05	......	......	......	5.88	5.93	......	......	......	....

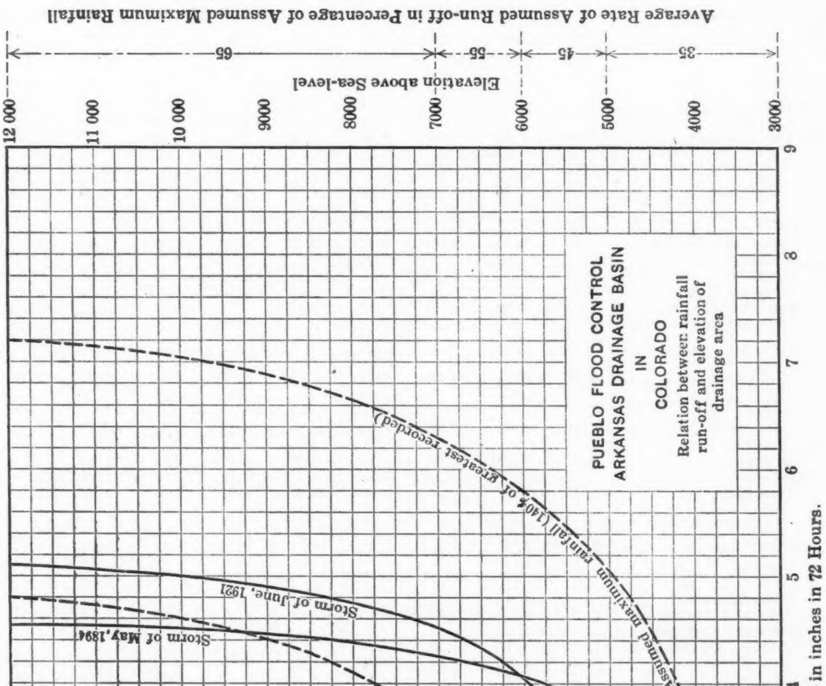

Average Rate of Assumed Run-off in Percentage of Assumed Maximum Rainfall
65
55
45
35
Elevation above Sea-level
12 000
11 000
10 000
9000
8000
7000
6000
5000
4000
3000
PUEBLO FLOOD CONTROL
ARKANSAS DRAINAGE BASIN
IN
COLORADO
Relation between rainfall run-off and elevation of drainage area
Assumed maximum rainfall (140% of greatest recorded)
Storm of June, 1921
Storm of May, 1894
Average rainfall
Average rainfall
Assumed maximum run-off
0
1
2
3
4
5
6
7
8
9
Rainfall in inches in 72 Hours.

FIG. 10.

TABLE 6.—DEPTH OF RAINFALL, IN INCHES, FOR VARIOUS

Elevation, in feet.	Station.	May, 1894.			July, 1899.			June, 1903.		
		24 Hours.	48 Hours.	72 Hours.	24 Hours.	48 Hours.	72 Hours.	24 Hours.	48 Hours.	72 Hours.
3 000 to 4 000	Holly	......	......	......	1.16	1.16	1.16	3.25	3.92	4.19
	Lamar	......	......	......	2.30	2.40	2.49	1.00	1.86	2.20
	Las Animas	1.09	1.16	1.16	1.95	2.07	2.07	1.20	1.60	1.60
	Average	1.09	1.16	1.16	1.80	1.88	1.91	1.82	2.46	2.66
4 000 to 5 000	Cheyenne Wells	2.00	2.00	2.00	0.37	0.52	0.69	0.25	0.40	0.53
	Ordway	......	......	......	......	......	......	......	......	......
	Pueblo	......	......	......	......	......	......	......	......	......
	Rocky Ford	3.50	3.50	3.50	2.08	2.55	2.70	1.65	1.74	1.90
	Springfield	4.00	4.00	4.10	......	......	......	......	......	......
	Two Buttes	......	......	......	......	......	......	......	......	......
	Average	3.17	3.17	3.20	1.23	1.54	1.70	0.95	1.07	1.22
5 000 to 6 000	Canon City	4.31	5.06	5.06	0.45	0.75	0.79	0.64	0.89	1.37
	Florence	......	......	......	......	......	......	......	......	......
	Hamps	2.00	2.70	2.76	0.18	0.32	0.36	0.92	1.84	2.18
	Hoehne	......	......	......	0.34	0.51	0.84	1.31	1.81	1.96
	Trinidad	......	......	......	0.66	0.87	1.01	1.10	1.59	2.35
	Average	3.16	3.88	3.91	0.41	0.61	0.75	0.99	1.53	1.97
6 000 to 12 000	Calhan	......	......	......	......	......	......	......	......	......
	Colorado Springs	2.95	4.39	4.47	1.03	1.18	1.29	1.27	2.13	2.41
	Cuchara Camps	......	......	......	......	......	......	......	......	......
	Divide Exp. Station	1.65	3.47	3.49	......	......	......	......	......	......
	Glen Eyrie	3.13	4.71	4.86	0.49	0.85	0.85	0.72	1.17	1.45
	Husted	1.15	1.99	2.07	......	......	......	0.75	1.50	1.65
	Lake Moraine	5.50	7.50	7.50	1.36	1.60	1.72	1.00	1.65	2.05
	Monument	......	......	......	......	......	......	......	......	......
	North Lake	......	......	......	......	......	......	......	......	......
	Salida	......	......	......	0.90	1.32	1.49	0.48	0.76	1.10
	Santa Clara	......	......	......	1.08	2.13	2.73	1.63	3.00	3.97
	Victor	......	......	......	......	......	......	......	......	......
	Wortman	......	......	......	......	......	......	......	......	......
	Average	2.88	4.41	4.48	0.97	1.42	1.62	0.98	1.70	2.10
3 000 to 12 000	General average	2.84	3.68	3.72	1.02	1.30	1.44	1.14	1.72	2.06

TABLE 7.

Elevation, in feet.	Average Rainfall, in Inches, in 72 Hours.	
	May, 1894.	June, 1921.
3 000 to 4 000	1.16	0.71
4 000 to 5 000	3.20	2.27
5 000 to 6 000	3.91	3.74
6 000 to 12 000	4.48	4.96
Average, 3 000 to 12 000	3.72	3.43

STATIONS IN THE ARKANSAS DRAINAGE AREA IN COLORADO.

Elevation, in feet.	Station.	JULY, 1914.			AUGUST, 1916.			JUNE, 1921.		
		24 Hours.	48 Hours.	72 Hours.	24 Hours.	48 Hours.	72 Hours.	24 Hours.	48 Hours.	72 Hours.
3 000 to 4 000	Holly	0.70	0.86	1.06	1.94	2.14	2.14	....	....	....
	Lamar	1.00	1.25	1.25	2.05	2.65	3.05	0.65	0.65	1.15
	Las Animas	0.77	1.17	1.17	2.15	2.25	2.27	0.27	0.27	0.27
	Average	0.82	1.09	1.16	2.05	2.35	2.49	0.46	0.46	0.71
4 000 to 5 000	Cheyenne Wells	0.20	0.20	0.20	1.48	1.48	1.48	....	....	....
	Ordway	....	....	....	....	....	....	0.90	1.65	1.84
	Pueblo	1.25	1.84	1.94	1.39	2.08	2.08	1.64	3.09	4.21
	Rocky Ford	2.00	2.32	2.61	2.50	2.58	3.08	1.40	2.20	2.35
	Springfield	....	....	....	....	....	....	....	....	....
	Two Buttes	0.66	1.07	1.48	2.87	3.14	4.75	0.22	0.37	0.67
	Average	1.03	1.36	1.56	2.06	2.32	2.85	1.04	1.83	2.27
5 000 to 6 000	Canon City	1.10	1.60	1.75	0.90	1.08	1.08	2.35	3.10	3.40
	Florence	....	....	....	....	....	....	3.31	5.78	6.77
	Hamps	0.78	1.08	1.08	1.36	1.36	1.36	....	....	....
	Hoenne	1.20	1.81	2.11	0.80	0.95	1.16	....	....	....
	Trinidad	0.75	1.53	2.06	0.97	0.97	0.97	0.55	0.85	1.05
	Average	0.96	1.51	1.75	1.01	1.09	1.14	2.07	3.24	3.74
6 000 to 12 000	Calhan	....	....	....	....	....	....	3.26	4.09	4.48
	Colorado Springs	2.00	2.03	2.03	0.28	0.28	0.28	5.00	9.40	10.66
	Cuchara Camps	....	....	....	....	....	....	0.86	1.07	1.19
	Divide Exp. Station	....	....	....	....	....	....	....	....	....
	Glen Eyrie	....	....	....	....	....	....	....	....	....
	Husted	....	....	....	....	....	....	....	....	....
	Lake Moraine	0.44	0.77	1.17	0.85	1.43	1.75	3.68	4.33	5.73
	Monument	....	....	....	....	....	....	2.90	3.72	3.72
	North Lake	0.78	1.46	2.01	0.75	1.37	1.57	....	....	....
	Salida	0.80	0.94	1.04	0.72	1.42	1.42	....	....	....
	Santa Clara	....	....	....	....	....	....	....	....	....
	Victor	....	....	....	....	....	....	2.08	3.63	4.00
	Wortman	0.40	0.60	0.70	1.00	1.00	1.00	....	....	....
	Average	0.88	1.16	1.39	0.72	1.10	1.20	2.70	3.63	4.96
3 000 to 12 000	General average	0.93	1.28	1.48	1.37	1.64	1.84	1.94	2.95	3.43

Curves showing the average rainfall in 72 hours at different elevations of the Arkansas River drainage area will be found on Fig. 10 for the storms of May, 1894, and June, 1921. Fig. 10 also shows a curve of assumed maximum rainfall in 72 hours, based on 140% of the greatest recorded rainfall in the storms of May, 1894, and June, 1921. It also shows a curve of assumed maximum run-off based on the following assumed percentages of run-off for the different elevations of the drainage area:

Elevation.	Percentage of Run-Off.
3 000 to 5 000	35
5 000 to 6 000	45
6 000 to 7 000	55
7 000 and over	65

TABLE 8.—PROBABLE MAXIMUM RUN-OFF FROM DRAINAGE AREA OF THE ARKANSAS RIVER BETWEEN CANON CITY AND PUEBLO (1 740 SQ. MILES), BASED ON THE STORMS OF MAY, 1894, AND JUNE, 1921.

(See Fig. 10)

Elevation above sea level, in feet.	AREA.		AVERAGE RAINFALL IN 72 HOURS.				RUN-OFF.		
	Square miles.	Acres.	Maximum recorded.		Probable maximum.		Assumed percentage of rainfall.	Based on maximum recorded rainfall, in acre feet.	Probable maximum, in acre-feet.
			Inches.	Acre-feet.	Inches.	Acre-feet.			
4 000 to 5 000	88	56 320	3.20	15 018	4.50	21 120	35	5 256	7 392
Sub-total..	88	56 320	......	15 018	......	21 120	......	5 256	7 392
5 000 to 6 000	632	404 480	3.91	131 793	5.50	185 387	45	59 307	83 424
Sub-total...	720	460 800	......	146 811	......	206 507	......	64 563	90 816
6 000 to 7 000	229	146 560	4.32	52 762	6.05	73 891	65	29 019	40 640
Sub-total...	949	607 360	......	199 573	......	280 398	.....	93 582	131 456
7 000 to 12 000	791	506 240	4.96	209 246	6.96	293 619	65	136 010	190 852
Total...	1 740	1 113 600	......	408 819	......	574 017	......	229 592	322 308*

NOTE.—The probable maximum rainfall is estimated to be 40% in excess of the greatest recorded rainfall during the storms of May, 1894, and June, 1921.

* Equivalent to 3½-in. run-off from 1 740 sq. miles.

Using the same storm area that contributed to the recent flood in the Arkansas River, namely, 1 740 sq. miles between Canon City and Pueblo, and applying the assumed maximum run-off rates, as shown on Fig. 10, to respective areas for different elevations of the storm area, a value of 322 000 acre-ft. has been obtained for the probable maximum run-off through Pueblo. The calculation of this quantity is shown in detail in Table 8. A run-off of 322 000 acre-ft. from 1 740 sq. miles is equivalent to an average run-off of 3½ in. Based on this total run-off, a hydrograph (Fig. 11) has been constructed of a shape similar to the hydrograph of the flood of June, 1921, which indicates a probable peak flow of about 168 000 sec-ft. in the Arkansas River through Pueblo. A similar study of the drainage area of Fountain Creek (Table 9) results in a probable maximum run-off of 164 000 acre-ft. and a probable peak flow of 110 000 sec-ft., as indicated on Fig. 12.

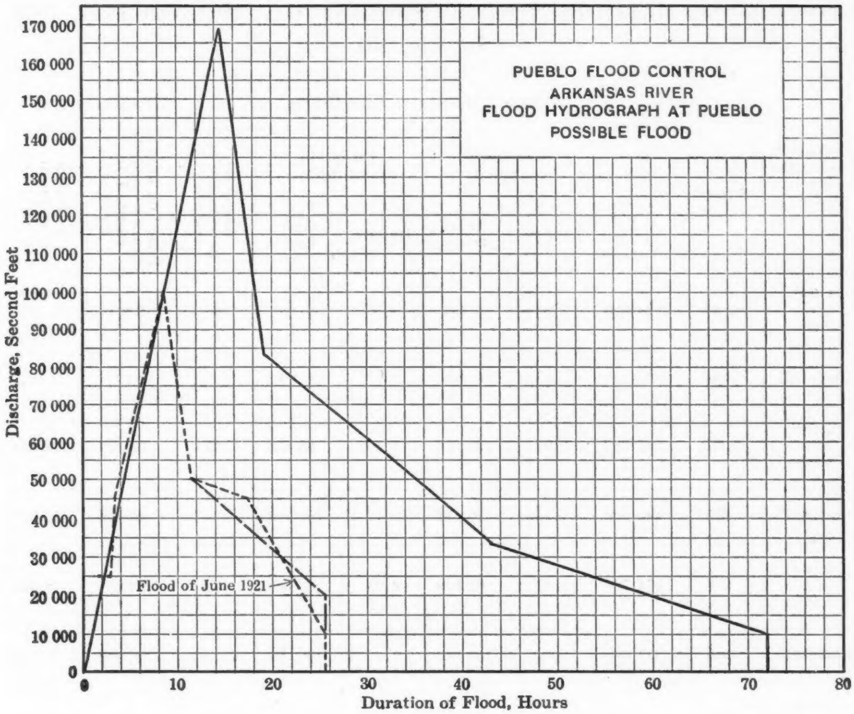

PUEBLO FLOOD CONTROL
ARKANSAS RIVER
FLOOD HYDROGRAPH AT PUEBLO
POSSIBLE FLOOD
Flood of June 1921
Discharge, Second Feet
Duration of Flood, Hours
0
10 000
20 000
30 000
40 000
50 000
60 000
70 000
80 000
90 000
100 000
110 000
120 000
130 000
140 000
150 000
160 000
170 000
10
20
30
40
50
60
70
80

FIG. 11.

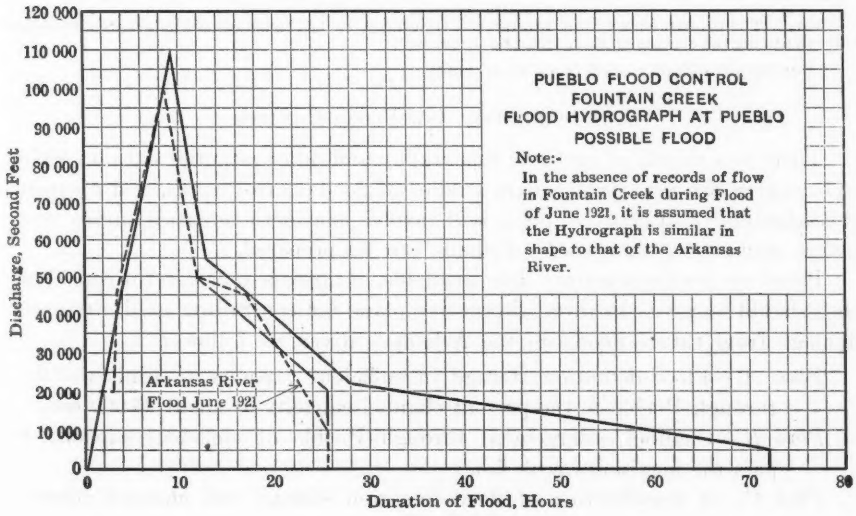

PUEBLO FLOOD CONTROL
FOUNTAIN CREEK
FLOOD HYDROGRAPH AT PUEBLO
POSSIBLE FLOOD
Note:-
In the absence of records of flow in Fountain Creek during Flood of June 1921, it is assumed that the Hydrograph is similar in shape to that of the Arkansas River.
Arkansas River
Flood June 1921
Discharge, Second Feet
Duration of Flood, Hours
0
10 000
20 000
30 000
40 000
50 000
60 000
70 000
80 000
90 000
100 000
110 000
120 000
10
20
30
40
50
60
70
80

FIG. 12.

TABLE 9.—PROBABLE MAXIMUM RUN-OFF FROM DRAINAGE AREA OF FOUNTAIN CREEK (930 SQ. MILES), BASED ON THE STORMS OF MAY, 1894, AND JUNE, 1921. (See Fig. 10)

Elevation above sea level, in feet.	AREA.		AVERAGE RAINFALL IN 72 HOURS,				RUN-OFF.		
	Square miles.	Acres.	Maximum recorded.		Probable maximum.		Assumed percentage of rainfall.	Based on maximum recorded rainfall, in acre-feet.	Probable maximum, in acre-feet.
			Inches.	Acre-feet.	Inches.	Acre-feet.			
4 000 to 5 000	38	24 320	3.20	6 485	4.50	9 120	35	2 270	3 192
Sub-total...	38	24 320	......	6 485	......	9 120	......	2 270	3 192
5 000 to 6 000	337	215 680	3.91	70 276	5.50	98 853	45	31 624	44 484
Sub-total...	375	240 000	......	76 671	......	107 973	......	33 894	47 676
6 000 to 7 000	273	174 720	4.32	62 899	6.05	88 088	55	34 594	48 448
Sub-total...	648	414 720	......	139 660	......	196 061	......	68 488	96 124
7 000 to 12 000	282	180 480	4.96	74 598	6.96	104 678	65	48 478	68 041
Total..	930	595 200	......	214 258	......	300 739	......	116 966	164 165*

NOTE.—The probable maximum rainfall is estimated to be 40% in excess of the greatest recorded rainfall during the storms of May, 1894, and June, 1921.

* Equivalent to 3¼-in. run-off from 930 sq. miles.

RECONSTRUCTION AND FLOOD CONTROL

There is a dearth of accurate topography and other essential data on which to base even the most preliminary studies of flood-control works, and a careful investigation of alternative plans will involve considerable detail survey work before accurate designs and estimates can be prepared.

Based on the fragmentary data available, it appears that careful consideration should be given to three alternative plans for controlling or limiting the damage from future floods on the Arkansas River, as follows:

Plan A.—Flood-detention storage in sufficient capacity to limit the flow through Pueblo to the present channel capacity of about 25 000 sec-ft.;

Plan B.—Channel enlargement through Pueblo of sufficient capacity to pass the maximum peak flow;

Plan C.—A combination of flood-detention storage and channel enlargement.

Preliminary studies suggest the careful consideration of four alternative plans for limiting the damage from future floods in Fountain Creek, as follows:

Plan D.—Flood-detention storage;

Plan E.—Bank protection along both banks of the flood-plain;

Plan F.—Channel enlargement;

Plan G.—A combination of flood-detention storage with either bank protection or channel enlargement.

Flood Control on Arkansas River

Plan A.—A solution of the problem by flood-detention storage alone would involve the construction of one or more detention reservoirs of sufficient capacity to retard all waters in excess of the present channel capacity through Pueblo, estimated at 25 000 sec-ft. Based on the assumed maximum flood in the Arkansas River, represented in the hydrograph shown on Fig. 11, detention storage of 210 000 acre-ft. would be required to control the flow through Pueblo to 25 000 sec-ft.

The Rock Canyon Reservoir site, on the Arkansas River, about 8 miles west of Pueblo, is believed to be the best site on the river for a detention reservoir. This site is in a gorge in hard sandstone, which formation is known to be of great depth. The floor of the reservoir is privately owned and is partly under cultivation and partly in pasture. The tracks of the Denver and Rio Grande and Santa Fé Railroads, also the upper end of the Bessemer Canal, are in the reservoir site.

Utilization of this site would involve the reconstruction of these railroads for a distance of probably 8 to 10 miles and also some changes in the Bessemer Canal, unless it was considered safe to leave them in the reservoir area. In this connection, it is probable that a flood-detention storage reservoir with large permanent outlets could be utilized for the partial control of unusual floods without increasing the damage from such floods to the railroads or the Bessemer Canal in their present locations. In other words, the railroads and the Bessemer Canal are so located that they are subject to damage from great floods, the damage being principally due to high velocities in the river. With a flood-detention reservoir of large outlet capacity all ordinary floods would pass the dam without storing water, and only the most unusual floods would submerge the railroads and the Bessemer Canal by storing water in the reservoir. Under present conditions, the effect of submergence in comparatively quiet water might cause less damage than the high velocities.

It has been stated previously that a detention storage capacity of about 210 000 acre-ft. would be required to control the assumed maximum flood in the Arkansas River to a flow of 25 000 sec-ft. through Pueblo. Unfortunately, the available data indicate that the Rock Canyon Reservoir site cannot be economically developed to this capacity, and there are probably no other sites of suitable location and sufficient capacity to control the floods. It is concluded, therefore, that Plan *A* may not prove to be feasible, although it should be carefully considered when full data become available.

Fig. 13 shows a capacity curve and Fig. 14 a cost curve for the Rock Canyon Reservoir site from which the costs given in Table 10 are taken. Similarly, Figs. 15 and 16 show a capacity curve and a cost curve, respectively, for the Steel Hollow Reservoir site.

TABLE 10.—COSTS OF STORAGE IN ROCK CANYON RESERVOIR.

Height of dam, in feet.	Reservoir capacity, in acre-feet.	Estimated cost.
75	32 000	$2 000 000
100	61 000	4 000 000
125	100 000	6 500 000

It will be noted that storage in the Rock Canyon Reservoir site is estimated to cost about $65 per acre-ft. Storage on the tributaries, if available at all, would probably cost over 50% more than storage in the Rock Canyon Reservoir, or, say, $100 per acre-ft. On this basis it might be estimated that 210 000 acre-ft. of flood-detention storage, if available, would cost approximately, as follows:

100 000 acre-ft., at $65..........................	$6 500 000
110 000 acre-ft., at $100 (say)....................	11 000 000
Total	$17 500 000

Plan B.—A solution of the flood problem on the Arkansas River, based on channel enlargement through Pueblo alone (with no flood-detention storage), would involve the widening and deepening of the present river channel or the construction of other channels, in addition to extensive levee work. In the section of the paper dealing with the drainage area and rainfall data (page 176), it was shown that the peak flow resulting from the maximum assumed storm would be 168 000 sec-ft. This represents the required channel capacity through Pueblo under Plan *B*.

Preliminary studies based on incomplete data indicate that a concrete-lined channel (lined on both sides and the bottom), following in general the alignment of the present river channel, is likely to prove the most economical method of passing the peak flow through Pueblo. Such a channel with a capacity of 168 000 sec-ft., extending from above the Fourth Street Bridge to a point below the Missouri Pacific Bridge, together with the necessary levee construction, is estimated to cost $5 500 000, including excavation, concrete, new bridges, raising of railroad grades, and right of way, as the main items of cost.

Studies have been made on a smaller concrete-lined channel supplemented with grass channels, in the nature of sunken gardens, one on each side of the concrete channel. The additional cost for bridges and for right of way under this arrangement would probably bring the cost to about $7 000 000.

Under Plan *B*, the grade of all bridges should be raised to provide a minimum clearance of 10 ft. between the high-water surface and the lowest

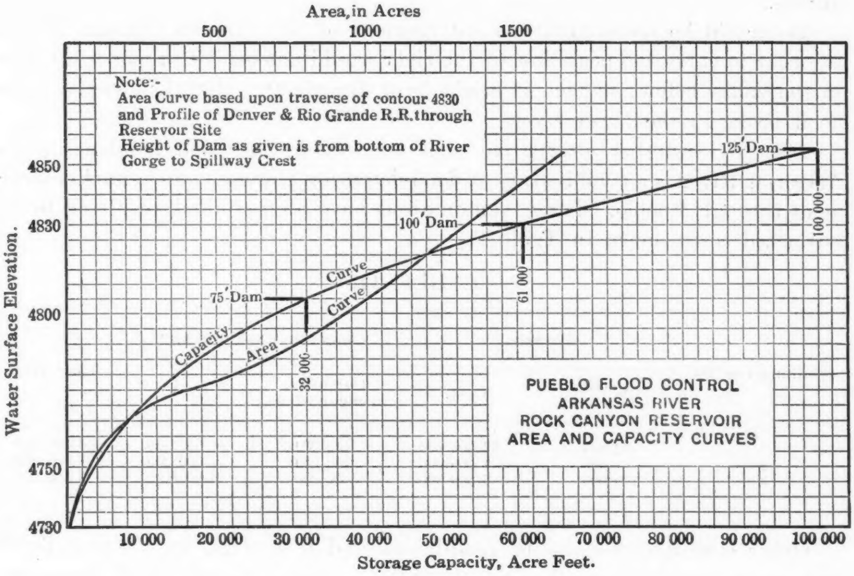

Area, in Acres
500
1000
1500
Note:-
Area Curve based upon traverse of contour 4830
and Profile of Denver & Rio Grande R.R. through
Reservoir Site
Height of Dam as given is from bottom of River
Gorge to Spillway Crest
125' Dam
100 000
100' Dam
61 000
Capacity Curve
Area Curve
75' Dam
32 000
PUEBLO FLOOD CONTROL
ARKANSAS RIVER
ROCK CANYON RESERVOIR
AREA AND CAPACITY CURVES
Water Surface Elevation.
4850
4830
4800
4750
4730
10 000
20 000
30 000
40 000
50 000
60 000
70 000
80 000
90 000
100 000
Storage Capacity, Acre Feet.

FIG. 13.

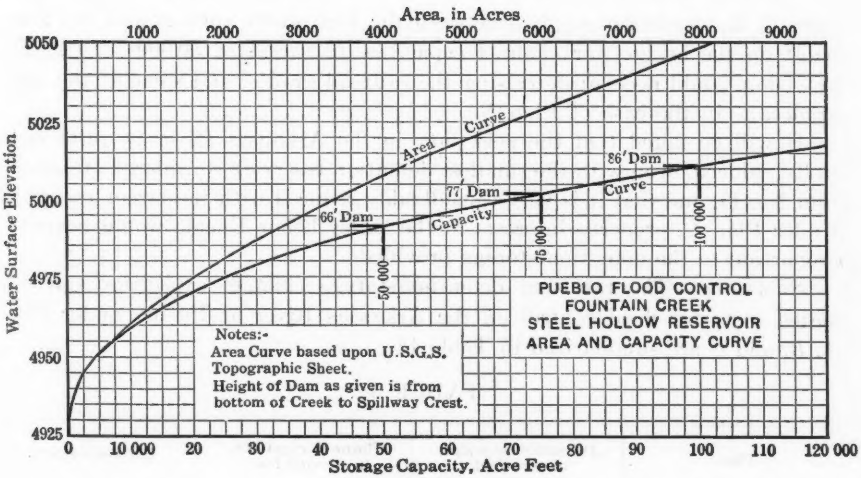

Area, in Acres
1000
2000
3000
4000
5000
6000
7000
8000
9000
Area Curve
86' Dam
77' Dam
Capacity Curve
66' Dam
50 000
75 000
100 000
PUEBLO FLOOD CONTROL
FOUNTAIN CREEK
STEEL HOLLOW RESERVOIR
AREA AND CAPACITY CURVE
Notes:-
Area Curve based upon U.S.G.S.
Topographic Sheet.
Height of Dam as given is from
bottom of Creek to Spillway Crest.
Water Surface Elevation
5050
5025
5000
4975
4950
4925
0
10 000
20
30
40
50
60
70
80
90
100
110
120 000
Storage Capacity, Acre Feet

FIG. 14.

point on the bridge. This provision would probably necessitate raising the railroad grades generally throughout the city, as well as some of the street grades.

It should be noted that the enlargement of the channel through Pueblo in order to carry the peak flow of the river would not benefit property interests in the valley below; in fact, it might have the effect of slightly increasing the flood peaks in the lower river.

Plan C.—Another alternative plan for solving the flood problem on the Arkansas River is a combination of flood-detention storage and channel enlargement through Pueblo. Three combinations have been studied, resulting in the estimated costs shown in Table 11.

TABLE 11.

Combination No.	Storage.		Channel Enlargement.		Total cost.
	Capacity, in acre-feet.	Cost.	Capacity, in second-feet.	Cost.	
1	32 000	$2 000 000	125 000	$4 500 000	$6 500 000
2	61 000	4 000 000	100 000	4 000 000	8 000 000
3	100 000	6 500 000	67 000	3 000 000	9 500 000

Other combinations can be readily selected from Fig. 19. It is believed that some combination like No. 1, with a comparatively small detention-storage reservoir and a large channel capacity, will prove to be the most economical.

Under Plan *C* the grade of all bridges should be raised in proportion to the channel capacity provided, and it is believed that a clearance of not less than 7½ ft. should be supplied between the high-water surface and the lowest point on the bridge, for channel capacities in excess of 50 000 sec-ft. This provision would necessitate raising the railroad grades, and some of the street grades, as in the case of Plan *B*.

It will be noted that the peak flow in the Arkansas River is quite materially reduced, even with the smallest detention reservoir considered (Combination No. 1), and that it is very considerably reduced with the larger reservoirs. Under Plan *C*, property interests in the valley below Pueblo would benefit in proportion to the detention storage provided.

Summary.—The resulting detention-storage channel capacities and estimated costs for flood control on the Arkansas River at Pueblo under Plans *A*, *B*, and *C* are summarized in Table 12.

TABLE 12.

Plan.	Detention storage, in acre-feet.	Channel capacity, in second-feet.	Estimated cost.
A........................	210 000	25 000	$17 500 000
B........................	None	168 000	5 500 000
C (Comb. No. 1)..........	32 000	125 000	6 500 000
C (Comb. No. 2)..........	61 000	100 000	8 000 000
C (Comb. No. 3)..........	100 000	67 000	9 500 000

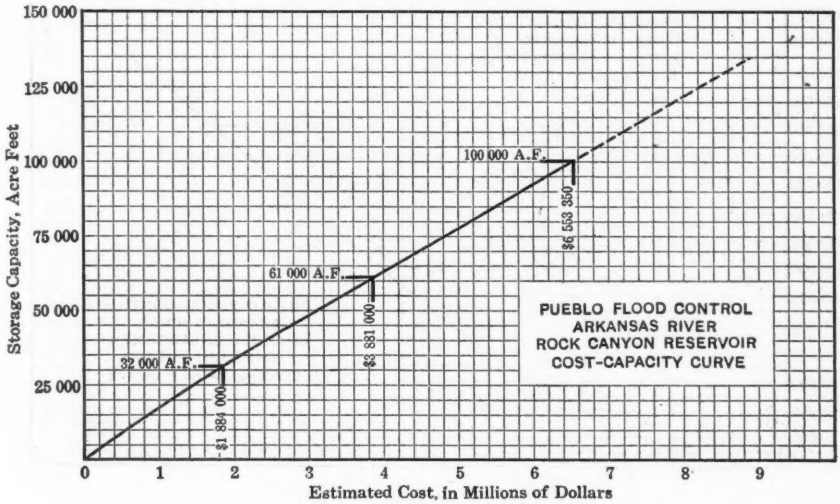

Storage Capacity, Acre Feet
150 000
125 000
100 000
75 000
50 000
25 000
100 000 A.F.
$6 553 350
61 000 A.F.
$3 881 000
32 000 A.F.
$1 884 000
PUEBLO FLOOD CONTROL
ARKANSAS RIVER
ROCK CANYON RESERVOIR
COST-CAPACITY CURVE
0
1
2
3
4
5
6
7
8
9
Estimated Cost, in Millions of Dollars

FIG. 15.

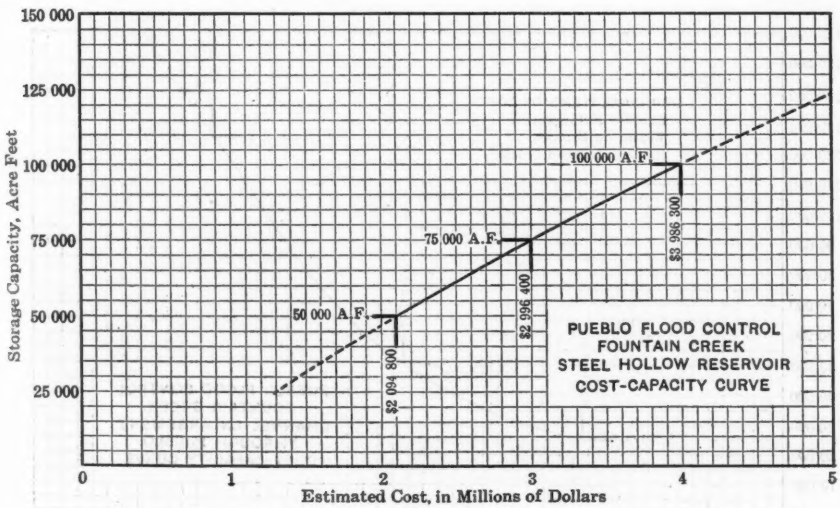

Storage Capacity, Acre Feet
150 000
125 000
100 000
75 000
50 000
25 000
100 000 A.F.
$3 986 300
75 000 A.F.
$2 996 400
50 000 A.F.
$2 094 800
PUEBLO FLOOD CONTROL
FOUNTAIN CREEK
STEEL HOLLOW RESERVOIR
COST-CAPACITY CURVE
0
1
2
3
4
5
Estimated Cost, in Millions of Dollars

FIG. 16.

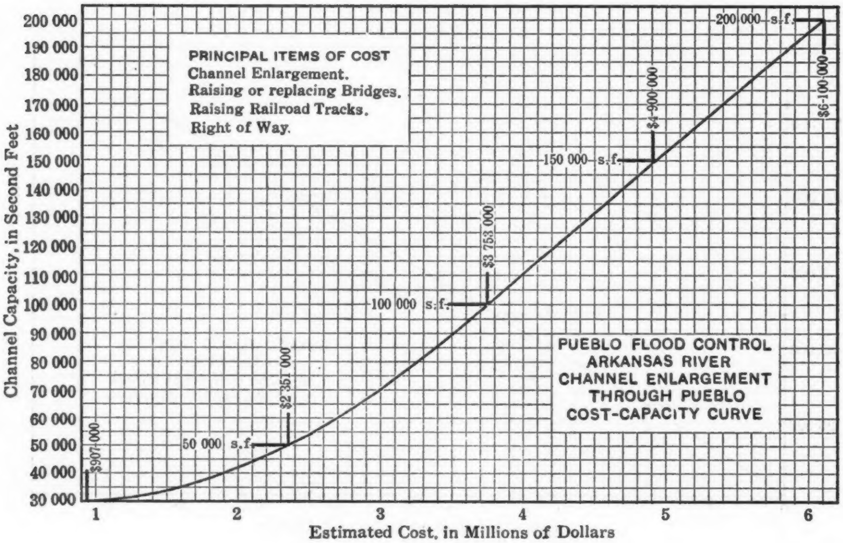

PRINCIPAL ITEMS OF COST
Channel Enlargement.
Raising or replacing Bridges.
Raising Railroad Tracks.
Right of Way.
PUEBLO FLOOD CONTROL
ARKANSAS RIVER
CHANNEL ENLARGEMENT
THROUGH PUEBLO
COST-CAPACITY CURVE
Channel Capacity, in Second Feet
Estimated Cost, in Millions of Dollars
200 000
190 000
180 000
170 000
160 000
150 000
140 000
130 000
120 000
110 000
100 000
90 000
80 000
70 000
60 000
50 000
40 000
30 000
1
2
3
4
5
6
$907 000
50 000 s.f.
$2 351 000
100 000 s.f.
$3 753 000
150 000 s.f.
$4 900 000
200 000 s.f.
$6 100 000

FIG. 17.

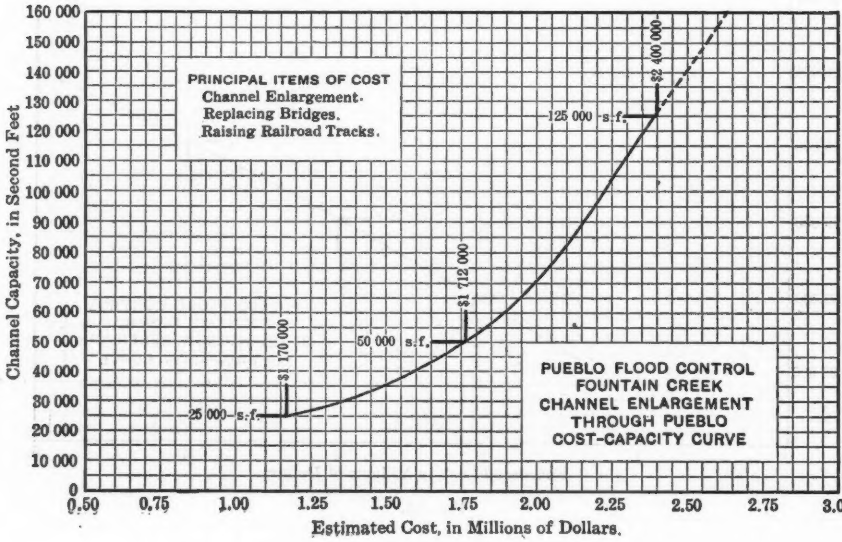
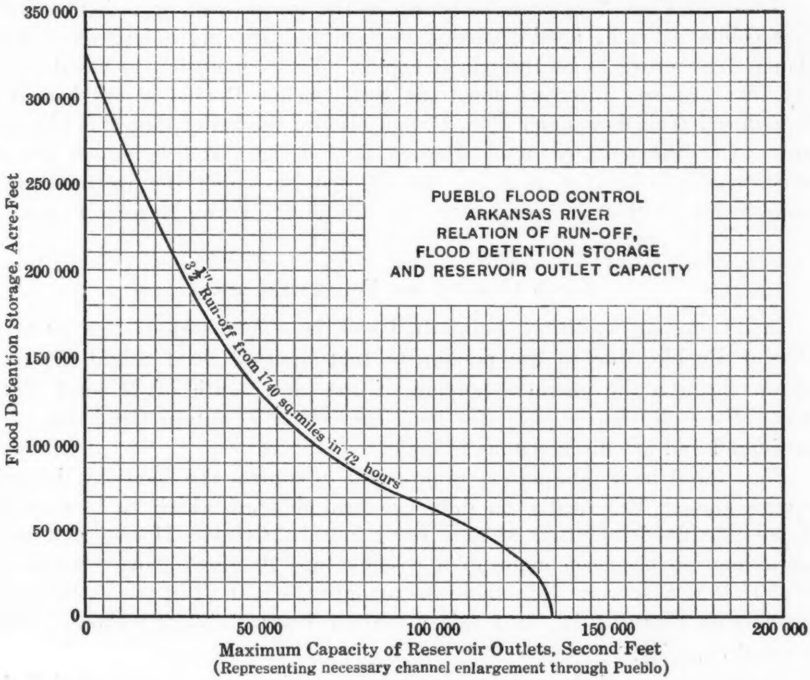
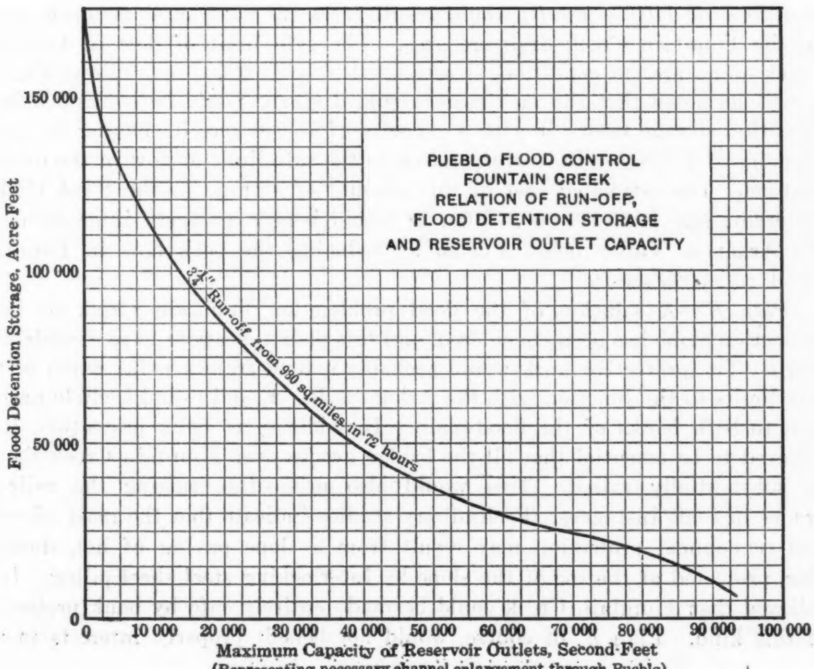

PRINCIPAL ITEMS OF COST
Channel Enlargement.
Replacing Bridges.
Raising Railroad Tracks.
PUEBLO FLOOD CONTROL
FOUNTAIN CREEK
CHANNEL ENLARGEMENT
THROUGH PUEBLO
COST-CAPACITY CURVE
Channel Capacity, in Second Feet
Estimated Cost, in Millions of Dollars.
160 000
150 000
140 000
130 000
120 000
110 000
100 000
90 000
80 000
70 000
60 000
50 000
40 000
30 000
20 000
10 000
0
0.50
0.75
1.00
1.25
1.50
1.75
2.00
2.25
2.50
2.75
3.00
25 000 s.f.
$1 170 000
50 000 s.f.
$1 712 000
125 000 s.f.
$2 400 000

FIG. 18.



PUEBLO FLOOD CONTROL
ARKANSAS RIVER
RELATION OF RUN-OFF,
FLOOD DETENTION STORAGE
AND RESERVOIR OUTLET CAPACITY
3¼" Run-off from 1740 sq.miles in 72 hours
Flood Detention Storage, Acre-Feet
350 000
300 000
250 000
200 000
150 000
100 000
50 000
0
0
50 000
100 000
150 000
200 000
Maximum Capacity of Reservoir Outlets, Second-Feet
(Representing necessary channel enlargement through Pueblo)

FIG. 19.



PUEBLO FLOOD CONTROL
FOUNTAIN CREEK
RELATION OF RUN-OFF,
FLOOD DETENTION STORAGE
AND RESERVOIR OUTLET CAPACITY
3¼" Run-off from 930 sq.miles in 72 hours
Flood Detention Storage, Acre-Feet
150 000
100 000
50 000
0
0
10 000
20 000
30 000
40 000
50 000
60 000
70 000
80 000
90 000
100 000
Maximum Capacity of Reservoir Outlets, Second-Feet
(Representing necessary channel enlargement through Pueblo)

FIG. 20.

Although Plan *B* appears to result in the lowest cost, it is believed that some combination of flood-detention storage and channel enlargement under Plan *C* may prove to be the wisest choice. This conclusion is based on the fact that property interests down the valley below Pueblo would benefit by the reduced peak flow under Plan *C,* and also on the belief that this plan may prove to be the most economical when final designs and estimates are prepared, particularly if a plan can be effected by which the railroad tracks would be left in their present location in the Rock Canyon Reservoir area.

Flood Control on Fountain Creek

In its present condition, Fountain Creek is a very serious menace to the City of Pueblo, due to the possibility that another flood might force the stream through the business portion of the city, south of Mineral Palace Park. Four alternative plans for flood control on Fountain Creek have been considered and are discussed, as follows:

Plan D.—Under this plan it is proposed to control the floods by detention storage alone. Practically no data are available on which to base studies of possible flood-detention storage on Fountain Creek. However, a rough approximation has been made of a possible site 10 miles above Pueblo (the Steel Hollow Reservoir site) where the topographical sheet of the U. S. Geological Survey (scale, $\frac{1}{2}$ in. = 1 mile; contour interval, 50 ft.) shows a basin of small capacity.

In the section of the paper dealing with drainage area and rainfall data (page 176), it has been estimated that a total volume of 164 000 acre-ft. and a peak flow of 110 000 sec-ft. would result from the maximum assumed storm on the Fountain Creek drainage area. The safe limit of flow in Fountain Creek under present conditions is estimated to be 25 000 sec-ft. From Fig. 20, it will be noted that the maximum assumed storm would be controlled by a detention storage reservoir with a capacity of 73 000 acre-ft., having an outlet capacity of 25 000 sec-ft., corresponding to the safe limit of flow in the present channel. The estimated cost of this amount of storage in the Steel Hollow Reservoir site is \$3 000 000. Plan *D* would benefit property interests down the Arkansas Valley below Pueblo by reducing the peak flow in Fountain Creek to 25 000 sec-ft.

Plan E.—A solution of the flood problem on Fountain Creek by bank protection alone has been considered, and this plan appears to have considerable merit. The protective work would probably extend from a point north of the city limits to the junction with the Arkansas River, and would include protection to both banks of the flood-plain. In addition to bank protection, it is believed to be essential that all the bridge grades over Fountain Creek should be substantially raised. This would also necessitate raising the railroad grades in some instances. Preliminary studies indicate that the most effective and economical protection may result from a slope paving of hot, dumped slag, protected at the toe of the slope by interlocking steel sheet-piling. It is believed that Fountain Creek could be made entirely safe by bank protection of this kind. Plan *E,* of course, would not benefit property interests in the

Arkansas Valley below Pueblo, inasmuch as it would have no effect in reducing the flood peaks from Fountain Creek.

Plan F.—Consideration has been given to the possibility of controlling the floods of Fountain Creek by confining the stream in a concrete-lined channel for a reach of about three miles above its junction with the Arkansas River. Plan *F* would also involve raising bridge and railroad grades, as in the case of Plan *E*. Rough estimates show that this plan would cost approximately $2 500 000 for a channel of sufficient capacity to carry the estimated peak flow of 110 000 sec-ft. This plan, it is believed, will warrant careful consideration. It should be noted that Plan *F* would not benefit property interests in the Arkansas Valley below Pueblo.

Plan G.—Another alternative plan for flood control on Fountain Creek is embraced in a combination of flood storage and either bank protection or channel enlargement. Two combinations of detention storage and channel enlargement have been considered, resulting in the costs shown in Table 13.

TABLE 13.

Combination No.	Storage.		Channel Enlargement.		Total cost.
	Capacity, in acre-feet.	Cost.	Capacity, in second-feet.	Cost.	
1	25 000	$1 500 000	68 000	$2 000 000	$3 500 000
2	50 000	2 000 000	38 000	1 500 000	3 500 000

Under Plan *G*, the bridge and railroad grades would be raised in proportion to the channel capacity provided. It will be noted that the peak flow in Fountain Creek is quite materially reduced, even with a small detention reservoir. A comparison of the costs under this plan with the cost under Plan *E* or Plan *F* suggests the probability that bank protection alone or channel enlargement alone will prove to be more economical than any combination with detention storage. Property interests in the Arkansas Valley below Pueblo would be benefited in proportion to the reduction of the peak flow in Fountain Creek under Plan *G*.

Summary.—The resulting detention-storage channel capacities and estimated costs for flood control on Fountain Creek under Plans *D, E, F,* and *G*, are summarized in Table 14.

TABLE 14.

Plan.	Detention storage, in acre-feet.	Channel capacity, in second-feet.	Estimated cost.
D..........................	73 000	25 000	$3 000 000
E..........................	None	110 000	2 500 000
F..........................	None	110 000	2 500 000
G (Comb. No. 1)..........	25 000	68 000	3 500 000
G (Comb. No. 2)..........	50 000	38 000	3 500 000

Either Plan *D* or Plan *G* would provide considerable protection against flood damage to property interests down the Arkansas River below Pueblo, and the additional cost for this protection might be warranted. However, the estimated costs of these alternative plans are so close and the data on which they are based are so meager that definite conclusions are impossible as to the most attractive plan.

Irrigation Storage and Power Development

Preliminary consideration has been given to the possibility of utilizing a part of the storage capacity in the Rock Canyon Reservoir site or of providing other storage reservoirs for irrigation use. Some consideration has also been given to the possibility of developing a limited amount of power at the Rock Canyon Reservoir site for municipal purposes and utilizing some of the storage capacity in this reservoir for the domestic water supply of Pueblo.

The total acreage under irrigation in the Arkansas Valley in Colorado is about 450 000 acres. Complete records for sixteen years indicate that the average total run-off of the Arkansas River at Pueblo is 533 000 acre-ft. Rainfall records indicate an average rainfall of 0.86 ft. during the irrigation season from March to November, amounting to 388 000 acre-ft. over the irrigated area of 450 000 acres. The total average rainfall plus run-off at Pueblo is, therefore, 921 000 acre-ft., or only slightly more than 2 acre-ft. per acre, not allowing anything for canal losses by seepage and evaporation, which losses are known to be large. When it is considered that these figures are averages and that there are a great many years when the run-off is materially less, it is evident that the supply for additional irrigation development is very limited. If, however, ample storage capacity was available on the river above Pueblo at reasonable cost, it might prove feasible to develop hold-over storage in sufficient quantity to be of some value in augmenting the supply to present canals or in furnishing a supply to new acreage. This possibility should be given further study when full information is available, notwithstanding the fact that the information at hand indicates no feasible project.

It would not be possible, of course, to utilize any of the flood-detention storage capacity for irrigation storage, and any capacity reserved for irrigation use would have to be additional to the required detention storage. Any capacity developed for irrigation storage in conjunction with a detention reservoir would be at the bottom of the reservoir, and the permanent outlets for detention storage would be placed at the top of the irrigation storage. Irrigation storage located below the permanent outlets would be subject to comparatively rapid silting up and, in this connection, it is believed that the yearly loss by silting might amount to as much as 0.25% of the total average run-off, or about $0.25 \times 533\,000 = 1\,330$ acre-ft. per year. This loss by silting should be given consideration, particularly in the case of a small development for irrigation storage.

The possibility of utilizing a small amount of storage capacity in the Rock Canyon Reservoir site as a domestic supply for the City of Pueblo has been considered, but this plan does not appear to be feasible, due to the fact

that any small reservoir on the main channel of a silt-laden stream would soon be filled with silt. It is believed that the settling reservoir would have to be located off the main stream and fed by a canal similar to the plan now followed. Such an arrangement does not appear to be feasible in connection with the Rock Canyon Reservoir.

The possibility of developing power at the Rock Canyon Reservoir site would depend on whether or not irrigation storage was developed. If no irrigation storage was developed, and the permanent outlets of the detention reservoir were placed at the lowest point in the reservoir, as would be the case, there would be no head for power development. If irrigation storage was developed in any considerable quantity, there might be a possibility of developing a limited amount of power. However, it is believed that very little, if any, firm power would result from such an installation, due to the fact that the development of any considerable irrigation storage would involve holding the water over, possibly for several years at a time.

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RAINFALL AND RUN-OFF STUDIES

By C. E. Grunsky,* M. Am. Soc. C. E.

To be Presented October 5th, 1921.

Synopsis

In California it has been necessary, by force of circumstances, to use rainfall records to a very great extent when approximating the water production of water-sheds. There has been made in that State, therefore, perhaps more than anywhere else in the United States, a study of rainfall and of the relation between rainfall and run-off. As early as 1884, the writer, then Chief Assistant State Engineer, began the study of rain distribution throughout California and prepared for the State Engineer Department a rainfall map (published by the State but now out of print) based on some 200 rain-station records. All these records were expanded to a common 14-year period (1870 to 1884), for which period the greatest number of complete station records were available. A method of combining station records, after first expressing the rainfall of the climatic year in percentage of the normal annual rainfall, and a method of extending short-term records to long periods, are explained in this paper.

Data are presented to show the range of precipitation in climatic years in the central portions of California, and also the frequency of climatic years with various amounts of precipitation. The difference is pointed out between the normal run-off from any water-shed and the probable run-off that is to be expected in a single season in which the rainfall is normal. The effect of altitude on the intensity of rainfall and on the run-off is discussed, and it is shown that the lower temperature at high altitudes diminishes evaporation and, consequently, increases run-off.

Formulas are also presented for the calculation of maximum storm-water flow from small and from large areas after the maximum rain intensity for various time-periods has been ascertained. A brief reference to evaporation is made, and a table (Table 9) and a formula (Equation (47)) are presented for estimating the evaporation from known mean monthly temperatures.

*** San Francisco, Cal.**

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

The deduced formulas are:

Rain intensity:

$$I = \frac{C}{\sqrt{t}} \text{ in. per hour} \quad \ldots\ldots (1)$$

Maximum rainfall in 1 hour:

$$R = 0.129\ C \text{ in.} \quad \ldots\ldots (2)$$

Maximum urban storm-water flow:

$$d_m = 0.645\ a\ I \text{ sec-ft. per acre} \quad \ldots\ldots (17)$$

or,

$$d_m = \frac{5aR}{\sqrt{t}} \text{ sec-ft. per acre} \quad \ldots\ldots (19)$$

Maximum stream flow from large areas:

$$d'_m = 413\ a\ I \text{ sec-ft. per sq. mile} \quad \ldots\ldots (23)$$

or,

$$d'_m = \frac{3\ 200aR}{\sqrt{t}} \text{ sec-ft. per sq. mile} \quad \ldots\ldots (24)$$

$$a = \frac{60}{60 + c\ \sqrt[3]{t}} \quad \ldots\ldots (29)$$

$$d'_m = \frac{25\ 000\ I}{60 + c\ t^{0.33}} \text{ sec-ft. per sq. mile} \quad \ldots\ldots (30)$$

or,

$$d'_m = \frac{190\ 000\ R}{60\ t^{0.5} + c\ t^{0.83}} \text{ sec-ft. per sq. mile} \quad \ldots\ldots (31)$$

or, fairly approximate:

$$d'_m = \frac{C''\ R}{t^x} \text{ sec-ft. per sq. mile} \quad \ldots\ldots (34)$$

In the foregoing formulas make:

$c = 0.5$ for impervious areas;
$c = 5.0$ for mountainous areas;
$c = 20.0$ for rolling country;
$c = 50.0$ for flat country;
$c = 250.0$ for sandy regions;

$C'' = 3\ 500$ and $x = 0.5$ for impervious areas;
$C'' = 3\ 300$ and $x = 0.6$ for mountainous areas;
$C'' = 3\ 000$ and $x = 0.7$ for rolling country;
$C'' = 2\ 100$ and $x = 0.75$ for flat country;
$C'' = 600$ and $x = 0.8$ for sandy regions.

The Wet Season in California.

As the season advances and the fall months pass, the remark is heard on all sides, "I wonder what kind of a winter we are going to have." This implies, referring now to the Pacific Slope, and more particularly to California, a desire to know whether the winter will be "wet" or "dry"—a wet winter being one in which the rain and snow materially exceed the normal and a dry

winter one in which the precipitation is materially below the normal. To those who are not familiar with the climate of the Pacific Coast, it is necessary to explain that this region is not subject to thunder-storms during the spring and summer and that, in consequence, there is very little rain in the six months from May 1st to November 1st. About 90% of the seasonal rain falls in the other six months, with precipitation at its maximum in midwinter.

On the Atlantic Coast, the rainfall is distributed more uniformly throughout the twelve months of the year, the maximum occurring usually in the spring or midsummer months, and there is a corresponding difference in the behavior of the streams which are the recipients of the run-off resulting from rain or melting snow.

It should be recalled in this connection that many of the Western streams —referring to streams which are not fed, in large measure, from underground sources or by the melting of high altitude snowdrifts—go dry, or almost dry, during the summer and fall, although they may have a large flow in the winter and spring. Their seasonal flow, that is, their flow from some time in the fall, say, from October 1st to the end of the following September, is to be ascribed to the rainfall of the climatic or seasonal year. By climatic or seasonal year, the writer refers to the twelve months from some date about the middle or end of one, to the same date of the next, summer season. By common consent the rain-year of the Pacific Slope has come to be considered as beginning with July. This is proper, because the rain which falls in July and August is generally trifling in quantity and is negligible in its effect on run-off, while any rain in September should be counted as affecting the stream flow in the run-off year beginning about the end of that month. For Western conditions, the necessity is thus apparent of discussing the seasonal-annual, or the climatic-year, rainfall in its relation to seasonal annual run-off. A comparison of the calendar-year stream flow with the calendar-year rainfall would be meaningless. Likewise, any comparison of rainfall in one calendar year with that in another calendar year is not only valueless, but absurd.

To illustrate this point, let five consecutive climatic years, Fig. 1, be considered, in the first of which the rainfall has been normal; in the second year, 50% of normal; in the third, 200%; in the fourth, 50%; and in the fifth, again normal. For such seasons, the corresponding stream flow may have been about 100% of normal in the first and fifth seasonal periods of twelve months; about 25% of normal in the second and fourth periods; and 400% in the third period. By calendar years, assuming like quantities of rain each winter before and after January 1st, one-half of the rainfall of the first seasonal year would be combined with one-half of that of the second year, and so on, and, for four calendar years, the rainfall record would appear as 75% of the normal for two years thereof and 125% of normal for the other two. These figures, showing a departure of 25% from normal, instead of 50 to 100%, as in the case of the climatic years, do not give a correct idea of what has happened, and they cannot be brought into any instructive relation to the resulting run-off from the water-shed on which this rain fell. Any deductions attempted from them pertaining to the relation between annual run-off and rainfall would be misleading.

It is to be hoped that some day this fact will be recognized by the U. S. Weather Bureau, and that the publication of calendar-year precipitation totals will be discontinued with a substitution therefor of climatic-year totals. On the Atlantic Slope, non-compliance with this desirable subdivision of the calendar year has not resulted in the same degree of inconvenience to the engineer as in the West where the need therefor is more obvious. If uniformity of publication is desired, the requirements of the West in this important matter should control. In this respect the Weather Bureau should follow the example of the Water Resources Branch of the U. S. Geological Survey which has long adopted a run-off year beginning with October 1st, a time when throughout the country the streams are ordinarily at their low stages. The climatic year for which meteorological data are desired, should begin at any convenient time between July 1st and October 1st. As run-off due to rainfall

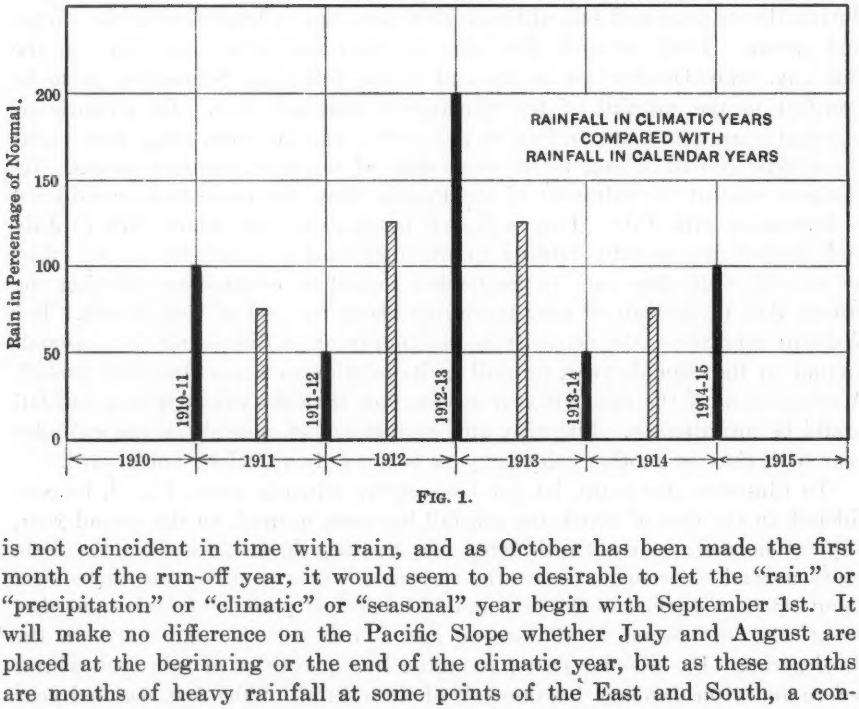

FIG. 1.

is not coincident in time with rain, and as October has been made the first month of the run-off year, it would seem to be desirable to let the "rain" or "precipitation" or "climatic" or "seasonal" year begin with September 1st. It will make no difference on the Pacific Slope whether July and August are placed at the beginning or the end of the climatic year, but as these months are months of heavy rainfall at some points of the East and South, a concession would be readily made, as indicated, to have them placed at the end of the rain-year instead of at its beginning, as is now the practice in California. This, in fact, was the plan, many years ago, of the State Engineer of California, whose compilation of rainfall data was published, in 1886, by the State of California in "Physical Data and Statistics," for climatic or rain-years beginning with September.

If the Weather Bureau declines to make such an innovation, it should at least publish the annual tables applying to Pacific Coast stations in a form made convenient for use by the addition of a column to its annual tables, in

which the totals for the rain-year terminating with June or with August are given.

This paper is intended to point the way to a better understanding of rainfall records and their application to a determination of the water resources and run-off phenomena of any particular region. In order to avoid misunderstandings, however, it will not be amiss to repeat that it has special application to the rainfall conditions of California, which in some essentials are different from those of the East. Thus, for example, it is generally known that wherever in this State the normal annual rainfall (this term being used to include melted snow) is upward of 10 in., an occasional minimum seasonal rainfall of about 30% of the normal (rarely less than 40%) is to be expected and that the maximum may be placed at about 200 per cent. On the Atlantic Coast, no such extreme variation has been observed. The ordinary range there is from about 25% below the annual normal to about 25% above the normal.

California is fortunate in having quite a number of long-time rainfall records, well scattered throughout the State, which are great aids in expanding the knowledge of rainfall to those parts of the State in which the local records cover only short time-periods. Reference should be made in this connection to an earlier paper* on this subject by the writer, dealing with rainfall in the San Francisco Bay region, in which it was explained that one of the characteristics of the cyclonic disturbances which bring rain to the Pacific Slope is the vastness of their extent. The storm the center of which takes a course across Oregon, or even British Columbia, may be accompanied by rainfall as far south as the southern boundary of California. One cyclonic disturbance follows another at intervals of a week or two, but not all of them bring rain. In "dry" winters, that is, in winters of less than normal rainfall, the storm path seems to persist somewhat more to the north than in winters in which the rainfall exceeds the normal. When the causes which fix the general position of the seasonal storm track are discovered, it may be possible to tell in advance whether the winter will be "wet" or "dry."

It is not only a fact that in California the rainfall which produces run-off in material amount is concentrated in the six months from November to the following April, but even during this period, which is frequently referred to as the "wet season", the fair days are more numerous than the rainy ones. Thus, for example, a curve has been constructed to show the frequency of rain for San Francisco. Although this study of rain frequency was made 20 years ago† the result may be accepted as applying to-day. It was found, as shown graphically in Fig. 2, that the average number of days per climatic year (12 months):

With some rain is 66 days;
With more than 0.25 in. of rain is 28 days;
With more than 0.50 in. of rain is 16 days;
With more than 0.75 in. of rain is 10 days; and
With more than 1.00 in. of rain is 6 days.

* "Rain and Run-Off near San Francisco, California," *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 496.

† "Report on a Sewerage System for San Francisco," by C. E. Grunsky, Marsden Manson, and C. L. Tilton.

And that there may be expected:

A day with more than 2.00 in. of rain less than once a year;
A day with more than 3.00 in. of rain once in 5 years;
A day with more than 4.00 in. of rain once in 25 years; and
A day with 5 in. of rain very rarely.

Although such facts as these are of hardly more than local interest, nevertheless, they indicate the scope of studies relating to the weather and to rainfall, which may have a bearing on the water output of a water-shed, or on the maximum storm flow, and for this reason they have been referred to here.

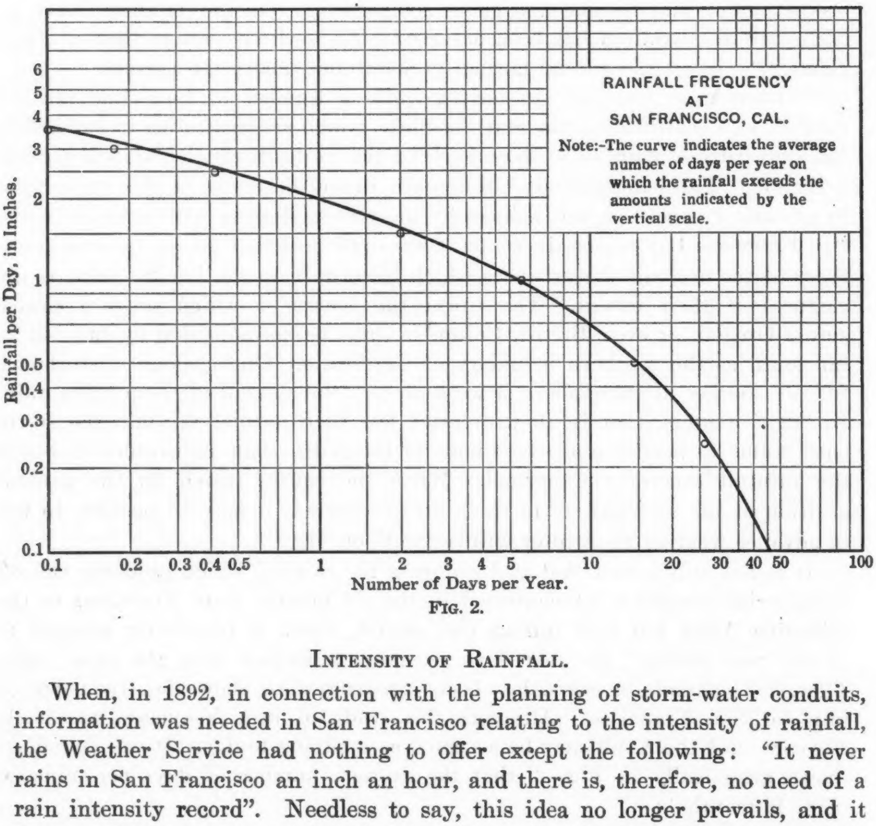

FIG. 2.

INTENSITY OF RAINFALL.

When, in 1892, in connection with the planning of storm-water conduits, information was needed in San Francisco relating to the intensity of rainfall, the Weather Service had nothing to offer except the following: "It never rains in San Francisco an inch an hour, and there is, therefore, no need of a rain intensity record". Needless to say, this idea no longer prevails, and it is now well known that although 1 in. of rain in 1 hour is highly improbable in San Francisco, there are short periods in which that rate of rainfall is greatly exceeded. For any place in the United States, it may be assumed (as amply verified by records of heavy rainfall) that the average rate of rainfall is inversely proportional to the square root of the time to which this rate applies. The following formula, therefore, will be found to give a dependable value of the maximum average rain intensity to be expected in any time-period during which there is heavy rainfall, when observations are

available which disclose the maximum quantity of rain which has fallen in the severest storms in some comparatively short time-period, such as in one-half hour, one hour, or even in several hours.

Let I = the maximum average rain intensity during t min., in inches per hour.

t = the time, in minutes, to which the rain intensity, I, applies;

R = the maximum quantity of rain to be expected in 1 hour; and

C = a coefficient of definite value for any locality, but not of the same value for different localities.

Then, for maximum intensity:

$$I = \frac{C}{\sqrt{t}} \text{ in. per hour} \qquad (1)$$

For maximum rainfall in 1 hour:

$$R = \frac{C}{\sqrt{60}} = \frac{C}{7.75} = 0.129\ C \text{ in.} \qquad (2)$$

For intensity coefficient:

$$C = I \sqrt{t} \qquad (3)$$

or, for maximum intensity:

$$I = \frac{7.75\ R}{\sqrt{t}} \text{ in. per hour} \qquad (4)$$

or, for intensity coefficient:

$$C = 7.75\ R \qquad (5)$$

The value of R, that is, the maximum fall of rain in 1 hour, is generally known from observation during heavy downpours. It is, therefore, a simple matter to determine the numerical value of C by Equation (3) or Equation (5) and, with Equation (1), to construct a curve of maximum intensity from which the maximum average rate of rainfall for any length of time, even to 24 hours and longer periods, can be scaled off. To have certain single-station records depart materially from the curve need not be disturbing, because such records are subject to unavoidable error, depending on the direction, force, and character of the wind, uniformity of exposure of the rain gauge to all points of the compass, and other like causes.

It may be noted that, for rainfall conditions similar to those in San Francisco, the value of C in Equation (1) is about 5 and for conditions similar to those in New York City, it is about 15. The maximum rainfall in 1 hour throughout any considerable part of San Francisco will rarely exceed 0.60 in. and in New York City, it will rarely exceed 2 in.

The diagram, Fig. 3, in which the relation between the maximum average intensity of rainfall for various time-periods, from 1 min. to 5 000 min., and for a number of values of the intensity coefficient, C, is shown, will be found to be helpful first, in determining the value of C when the maximum rain in 1 hour, or some other time-period, is known; and, thereafter, in determining by the aid of this value of C, the maximum intensity of rain in any number

of minutes. By using logarithmic scales, the lines of intensity of rainfall in Fig. 3, have been made to appear as straight, parallel lines.

It is noteworthy that Fig. 3 lends itself to a modification of the formula for rainfall intensity, Equation (1), which has determined the slope of the intensity lines. If any set of dependable results of observations for various time-periods are platted on the diagram, it may be found to be desirable to give the line limiting the maximum intensities a somewhat different slope from that which has resulted from the use of the factor, $\sqrt{t}$, in the denominator of Equation (1). From any slope thus ascertained, the preferred exponent of t, other than 0.5, can be readily ascertained. It would be well, however, to bear in mind that, owing to the many uncertainties which enter into the determination of the total quantity of rain on an area, this being at variance with single-point observations, there is good reason for the adoption of a formula of the simplest kind. In the same connection, it is to be assumed that records of rainfall applying to points and accepted as applying to areas are more dependable for the longer time-periods, that is, 30 min., 1 hour, or 2 hours, than for the 1-min., 5-min., and 10-min. periods.

In studying the results of observations of rain intensity applying to a water-shed, it should be remembered:

1.—That the rain-gauge record represents only what is happening at the particular point where the rain guage is placed.

2.—That at best the rain-gauge record is only a close approximation of what occurs a few feet away.

3.—That the average of the records of a number of rain gauges will give a better idea of what occurs during the passage of a storm over any place than will be given by a single gauge.

4.—That the maximum rates of rainfall during a storm at various points on a water-shed, no matter how small, do not occur at exactly the same time.

5.—That the maximum rate of rainfall applying to an entire water-shed is necessarily less than that applying to some points within the water-shed.

6.—That the distribution of rain to the various parts of a water-shed is not identical for all storms.

7.—That the mass curves of rain for any storm at various points of a water-shed will show variations in character, that is, sequence of intensities, as well as in quantities.

It follows from such considerations that in the determination of the storm characteristics or rainfall intensities which will give maximum storm-water flow from any water-shed, the single-station record, with its occasionally exaggerated rates of rainfall, at its best, will give only an approximation of the value which applies to the entire water-shed. The plan of combining several station records and of smoothing out the irregular curves that would result from close adherence to such records, is to be recommended. Of course, the fact remains that the maximum possible storm is not likely to be among the particular storms which have come under observation. The records of what has occurred in the past are valuable only as indicating what is probable or possible in the future, and the more complete these records are, the more dependable will be the conclusions which are based thereon.

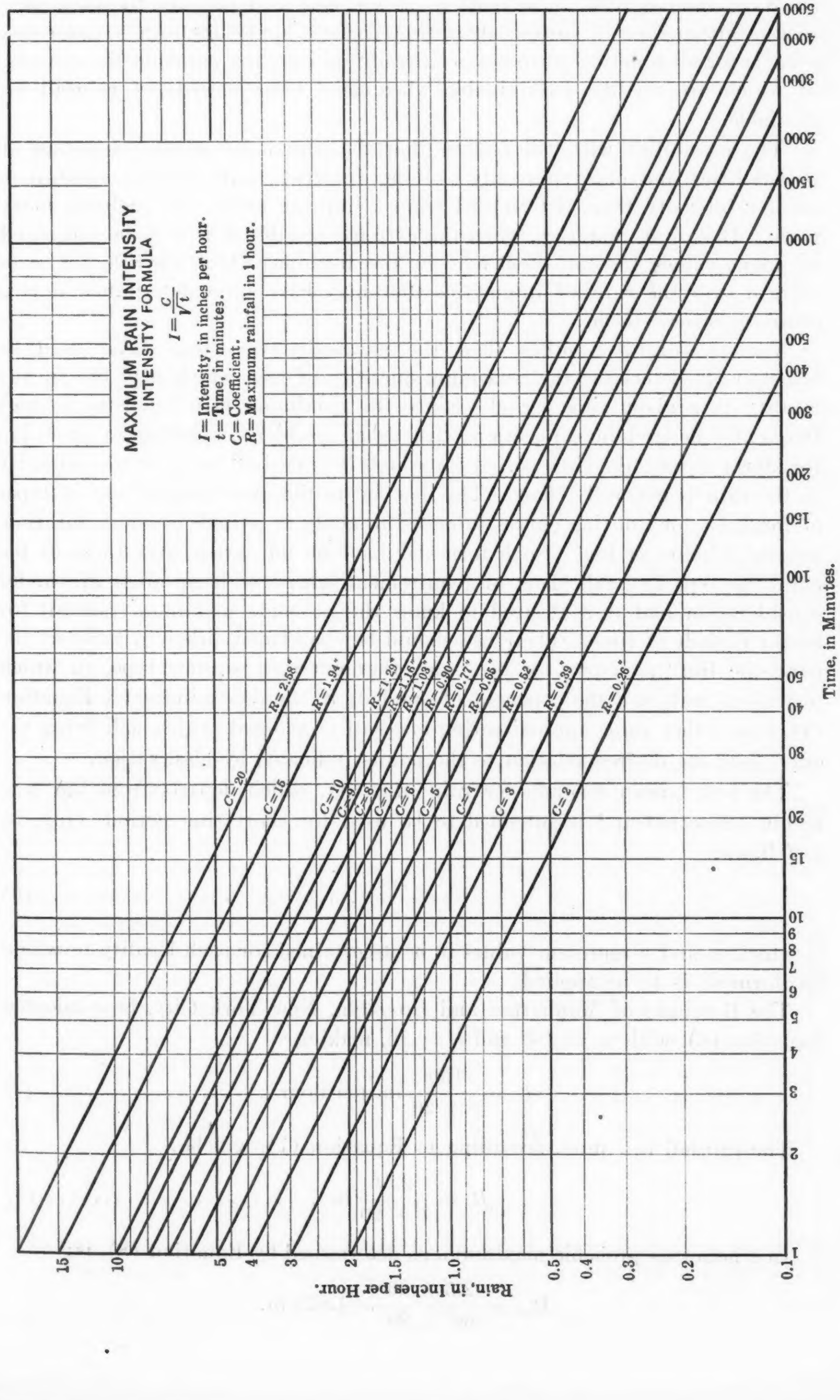
MAXIMUM RAIN INTENSITY
INTENSITY FORMULA
$I=\frac{C}{\sqrt{t}}$
I = Intensity, in inches per hour.
t = Time, in minutes.
C = Coefficient.
R = Maximum rainfall in 1 hour.
C=20
C=15
C=10
C=9
C=8
C=7
C=6
C=5
C=4
C=3
C=2
R=2.58"
R=1.94"
R=1.29"
R=1.16"
R=1.03"
R=0.90"
R=0.77"
R=0.65"
R=0.52"
R=0.39"
R=0.26"
Rain, in Inches per Hour.
15
10
5
4
3
2
1.5
1.0
0.5
0.4
0.3
0.2
0.1
Time, in Minutes.
1
2
3
4
5
6
7
8
9
10
15
20
30
40
50
100
150
200
300
400
500
1000
1500
2000
3000
4000
5000

FIG. 3.

With due regard to these matters, it will be found that the limiting curve of rainfall maxima in various short time-periods, applicable to any water-shed area, is for all practical purposes, as already indicated, a parabola the elements of which are readily determinable when good local records of rainfall are available.

The possibility will still remain that the occasional storm, exceeding all probabilities and occurring only at long intervals, will produce rainfall of greater intensity than, determined from a limiting curve, the probable maximum. It is well, therefore, when the extreme possible flow is to be estimated, to adopt values for the coefficient in the formula, which provide for some margin over the rainfall intensities that may have been determined during ordinary severe storms.

Various formulas, other than the parabolic type, have been used by engineers to determine the maximum quantity of rain which may fall in any locality in a given time-period. Since the maximum rain intensity is most frequently desired in connection with urban problems, relating to provision for storm waters, the time-period for which it is desired to know the intensity of the rain is generally short. This has led to the quite general use of types of formulas for limiting curves intended to apply to periods of short duration, usually 2 hours or less. Such formulas have no advantage over those of the parabolic type and fall short of requirement whenever large areas are under consideration and it is desired to know the probable maximum rainfall for longer periods of time. Their use should be abandoned either in favor of the parabolic limiting curve, or of some other curve of similar type, in which instead of making t the exponent of one-half in the denominator of Equation (1), some other value approximating one-half is adopted, which will bring the curve into the desired relation to the points indicated by observation.

The best known formula for rain intensity, which departs from the type herein advocated and is intended to be used for short time-periods only, is, as follows:

$$I = \frac{m}{t + n} \quad \text{.........................} (6)$$

in which m and n represent values to be ascertained for each locality to which the formula is to be applied.

The Boroughs of Manhattan and Brooklyn, New York City, have adopted Equation (6), with $m = 150$ and $n = 20$, making

$$I = \frac{150}{t + 20} \text{ in. per hour} \quad \text{.....................} (7)$$

The rainfall in t min., according to Equation (7), will be:

$$R_t = \frac{2.5t}{t + 20} \text{ in} \quad \text{........................} (8)$$

In 1 hour, the probable maximum, as determined by Equation (8), is:

$$R_{60} = \frac{2.5 \times 60}{60 + 20} = 1.875 \text{ in.}$$

For this value on a parabolic limiting curve, it will be found that:

$$I = \frac{14.5}{\sqrt{t}} \quad \text{.............................} (9)$$

and

$$R_t = 0.242 \sqrt{t} \quad \text{.............................} (10)$$

The defects of the New York formula will appear from the comparison shown in Table 1.

TABLE 1.—COMPARISON OF THE NEW YORK FORMULA FOR MAXIMUM RAINFALL WITH THE PARABOLIC FORMULA*

From New York Formula, $I = \frac{150}{t + 20}$; from Parabolic Formula, $I = \frac{14.5}{\sqrt{t}}$.

Time, in minutes.	Rain Intensity, in Inches per Hour, Based on:		Maximum Rainfall, in Inches:	
	New York formula.	Parabolic formula.	New York formula.	Parabolic formula.
1	7.2	14.5	0.12	0.24
5	6.0	6.5	0.50	0.54
10	5.0	4.6	0.83	0.73
20	3.75	3.24	1.22	1.08
30	3.00	2.65	1.50	1.32
60	1.88	1.88	1.88	1.88
90	1.36	1.53	2.04	2.29
120	1.07	1.32	2.14	2.64
180	0.75	1.09	2.25	3.27
240	0.58	0.94	2.32	3.76
300	0.47	0.84	2.35	4.20
360	0.39	0.76	2.37	4.56
720	0.20	0.54	2.43	6.48
1 440	0.10	0.38	2.46	9.12

*It is probable that if the two formulas had been made to agree at some time-period in excess of 60 min., making the value of C smaller, the resulting parabolic curve would better fit New York conditions than with C at 14.5.

In the comparison shown in Table 1, the two limiting curves and the two intensity curves have been made to coincide at the 1-hour points. Perhaps if all records of heavy rainfall at New York for time-periods up to 24 hours had been taken into account, a more appropriate parabolic curve for use in that vicinity could have been obtained. The trouble with any formula of the New York type is that it shows nearly as much rainfall for 2 or 3 hours as it does for 24 hours. In this respect, it is obviously defective.

If approximation by the New York type of formula to the maxima for short time-periods should be found to be closer at single-rainfall stations than by a parabolic formula, the question is still open as to whether the latter does not give a more reasonable and better approximation to what is taking place throughout an entire drainage basin the run-off of which is under study.*

RAINFALL IN THE CLIMATIC YEAR.

Returning now to rainfall in longer periods of time, such as a climatic year of 12 months, it has been found of great convenience in making com-

* "The Sewer System of San Francisco, and a Solution of the Storm-Water Flow Problem," *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 294.

parisons to substitute for inches of seasonal rain (that is, precipitation in a rain-year or 12 months), the relation in percentage which the rainfall of the particular 12 months in question bears to the normal rainfall. This is practicable whenever a few long-time rainfall records disclosing a dependable normal afford a good starting point. These records at what may be called primary base stations should cover a time-period of such length that a few additional seasons of either light or heavy rainfall would not materially change the value of the annual normals for these stations.

TABLE 2.—RAINFALL AT SAN FRANCISCO AND SACRAMENTO, CAL., FOR THE PERIOD 1849 TO 1919, EXPRESSED IN PERCENTAGE OF THE NORMAL SEASONAL RAINFALL

Normal annual rainfall at San Francisco = 22.7 in.
Normal annual rainfall at Sacramento = 19.0 in.

Climatic year, July 1st to the following June 30th.

Climatic year.	RAINFALL IN PERCENTAGE OF NORMAL:			Climatic year.	RAINFALL IN PERCENTAGE OF NORMAL:		
	San Francisco.	Sacramento.	San Francisco and Sacramento. Composite.		San Francisco.	Sacramento.	San Francisco and Sacramento. Composite.
(1)	(2)	(3)	(4)	(1)	(2)	(3)	(4)
1849-50........	146	189	168	1885-86........	139	170	155
1850-51........	33	25	29	1886-87........	83	74	79
1851-52........	81	95	88	1887-88........	74	61	68
1852-53........	155	191	176	1888-89........	105	105	105
1853-54........	105	106	106	1889-90........	202	178	190
1854-55........	104	98	101	1890-91........	78	83	80
1855-56........	96	72	84	1891-92........	81	80	81
1856-57........	88	55	72	1892-93........	96	126	111
1857-58........	97	79	88	1893-94........	81	89	85
1858-59........	97	85	91	1894-95........	113	127	120
1859-60........	99	119	109	1895-96........	94	122	108
1860-61........	86	82	84	1896-97........	102	91	97
1861-62........	217	187	202	1897-98........	41	55	48
1862-63........	60	63	62	1898-99........	74	79	77
1863-64........	45	41	43	1899-1900	81	106	94
1864-65........	108	118	113	1900-01........	93	106	100
1865-66........	101	94	98	1901-02........	84	91	88
1866-67........	153	133	143	1902-03........	80	87	84
1867-68........	171	172	172	1903-04........	91	89	90
1868-69........	94	85	91	1904-05........	103	116	110
1869-70........	85	71	78	1905-06........	91	126	109
1870-71........	62	44	53	1906-07........	114	126	120
1871-72........	153	121	137	1907-08........	76	64	70
1872-73........	80	73	77	1908-09........	112	115	114
1873-74........	105	123	114	1909-10........	86	59	72
1874-75........	81	95	88	1910-11........	112	116	114
1875-76........	115	138	127	1911-12........	62	50	56
1876-77........	44	47	46	1912-13........	53	42	48
1877-78........	137	123	130	1913-14........	130	111	120
1878-79........	108	91	100	1914-15........	121	91	106
1879-80........	116	131	124	1915-16........	119	96	108
1880-81........	121	129	125	1916-17........	70	68	69
1881-82........	69	80	75	1917-18........	51	56	54
1882-83........	87	89	88	1918-19........	113	91	102
1883-84........	140	124	132	1919-20........	46	47	47
1884-85........	80	87	84	1920-21........	102	89	96

Referring now to California, the records at San Francisco and at Sacramento, beginning in 1849, may be accepted as of this type. These records are presented in Table 2, the rainfall being expressed in percentage of normal. In Column (4) there is given a composite record, a combination of the two separate records, which is a fair index of the rainfall throughout a broad extent of central areas in California.

With the information contained in Table 2, all other rainfall records in the vicinity, beginning with those nearest, and those covering the greatest number of seasons, can be expanded to the full 70-year period of the base stations; and, thereupon, progressing into zones farther removed from the two primary base stations, by the use of secondary base stations, the remaining—often quite fragmentary—rainfall records of the State can likewise be expanded from short-time records of observed rainfall to long-time records of estimated rainfall. Thus, the normal annual rainfall can be determined for all places at which records have been kept. The percentage of normal rainfall noted for any place and season is, at once, an indication of the quantity of rain which fell in the vicinity of that place.

To illustrate the procedure, take Napa where the mean annual fall of rain during thirty-seven seasons, between 1877 and 1919, covered by records, was 24.3 in. The composite record in Table 2 shows that for these same years the mean annual rainfall was 103% of the normal rainfall. Consequently, the normal at Napa is $\frac{24.3}{1.03}$, or 23.6 in., and the Napa rainfall for each year of the 70-year period not covered by actual observation will be found by applying the percentage of normal in the composite column (Column 4) of Table 2 to the normal for this station. The results thus obtained will be practically as dependable as a basis for water production or run-off studies as if there had been actual measurement of rainfall at Napa during the full 70-year period. This is true, because a deduced record of this character, although departing more or less from what did actually take place, nevertheless, fairly represents the probable annual fluctuations in the rainfall and also quite dependably the range from probable minimum to probable maximum. It is possible in this fashion, as stated, to expand the rainfall records for any part of California and thus to produce for any region of the State a dependable long-time rainfall table generally covering the full period from 1849 to 1921.

Frequency of Wet and Dry Seasons.

It is interesting to study probabilities of seasonal rain intensity on the basis of the rainfall records of the past. Taking, for example, the composite record of the two California primary base stations, Table 2, and platting the same as ordinates in the order of their magnitude, a graph such as is shown in Fig. 4, will result.

An analysis of Fig. 4 shows that in 70 seasonal years, there probably will be no year with less than 30% of normal rain and that there will be:

2	years with	30 to 40%	of normal rain
2 to 3	years with	40 to 50%	of normal rain
3 to 4	" "	50 to 60%	" " "
5	" "	60 to 70%	" " "
7 to 8	" "	70 to 80%	" " "
9	" "	80 to 90%	" " "
9 to 10	" "	90 to 100%	" " "
9	" "	100 to 110%	" " "
7 to 8	" "	110 to 120%	" " "
4 to 5	" "	120 to 130%	" " "
2 to 3	" "	130 to 140%	" " "
2	" "	140 to 150%	" " "
3	" "	150 to 175%	" " "
2	" "	175 to 200%	" " "
70			

or, expanded to 100 years, to express probabilities in percentage, the curve will show that, in the region to which it applies, the rainfall in climatic years will range from a minimum of about 30% to a maximum of about 210% of the normal. In any considerable number of years, when grouped according to the rainfall, there will be:

Less than 30%	of normal rain in	0.5%	of all the years
30 to 40%	of normal rain in	1.1%	of all the years
40 to 50%	" " " "	3.3%	thereof
50 to 60%	" " " "	4.9%	"
60 to 70%	" " " "	7.3%	"
70 to 80%	" " " "	10.4%	"
80 to 90%	" " " "	13.1%	"
90 to 100%	" " " "	13.6%	"
100 to 110%	" " " "	12.9%	"
110 to 120%	" " " "	11.3%	"
120 to 130%	" " " "	6.6%	"
130 to 140%	" " " "	3.7%	"
140 to 150%	" " " "	2.6%	"
150 to 160%	" " " "	1.9%	"
160 to 170%	" " " "	1.6%	"
170 to 180%	" " " "	1.3%	"
180 to 200%	" " " "	1.9%	"
Over 200%	" " " "	1.4%	"

In a broad way it appears from the foregoing and from the diagram, Fig. 4, that in the central portions of California the precipitation in about 26.5% of the climatic years will be within 10% of the normal; that the precipitation in about 55% thereof, will be below normal; and that the precipitation in about 45% will be in excess of normal. It also appears that a climatic year

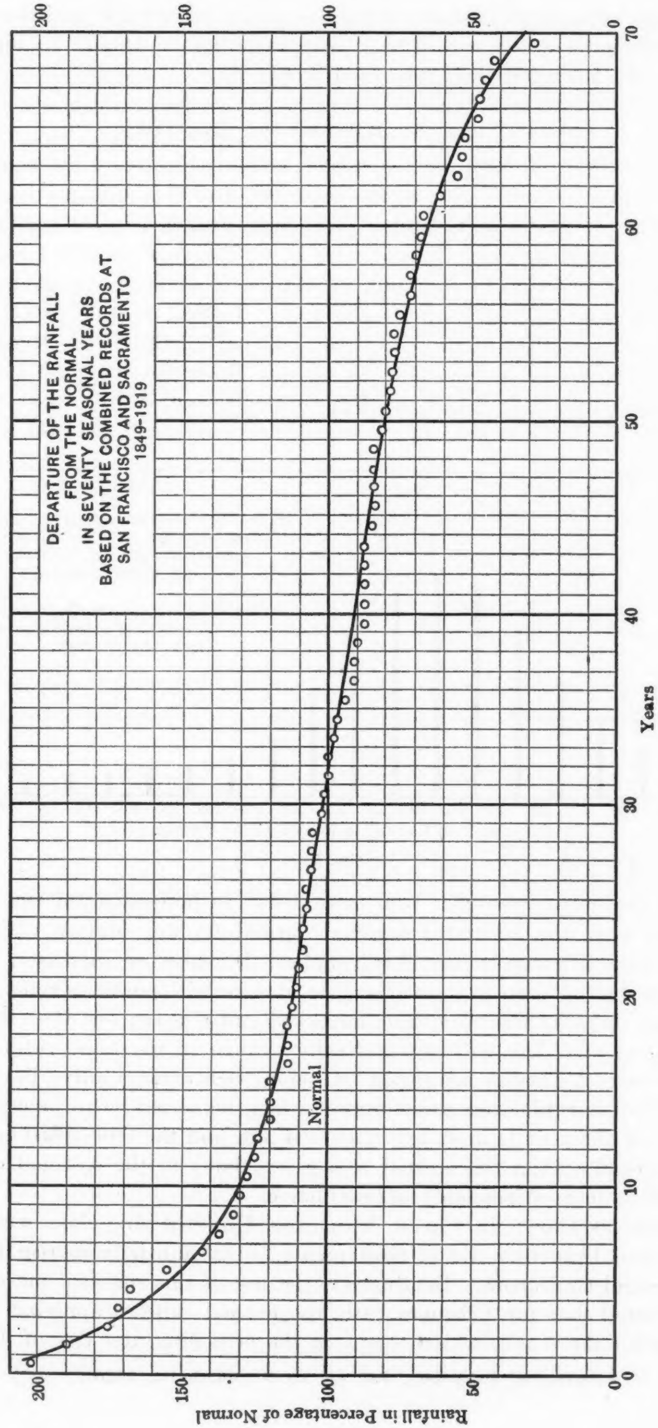
DEPARTURE OF THE RAINFALL
FROM THE NORMAL
IN SEVENTY SEASONAL YEARS
BASED ON THE COMBINED RECORDS AT
SAN FRANCISCO AND SACRAMENTO
1849-1919
Normal
Rainfall in Percentage of Normal
Years
200
150
100
50
0
10
20
30
40
50
60
70

FIG. 4.

with less rainfall than 30% of the normal is highly improbable and that there will be occasional seasons with 100% more rain than normal. In diagrammatic form, this information is presented in Fig. 5.

Normal Seasonal Rainfall to Any Date.

As each season or climatic year advances, a comparison with the rainfall to date of a normal season is frequently desirable, and this information, for a limited number of places in California, is being furnished to the public from day to day, through the press, by the local U. S. Weather Bureau forecasters. It has been the practice to compute, on the basis of past records for each station, the normal fall of rain to each date. Instead of this practice, the following graphic method of procedure for regional application is suggested.* For any station or for a composite as already described the mass curve of seasonal rain is platted, using monthly normals and calling the seasonal normal 100%, as shown in Fig. 6. The resulting curve will give for any day

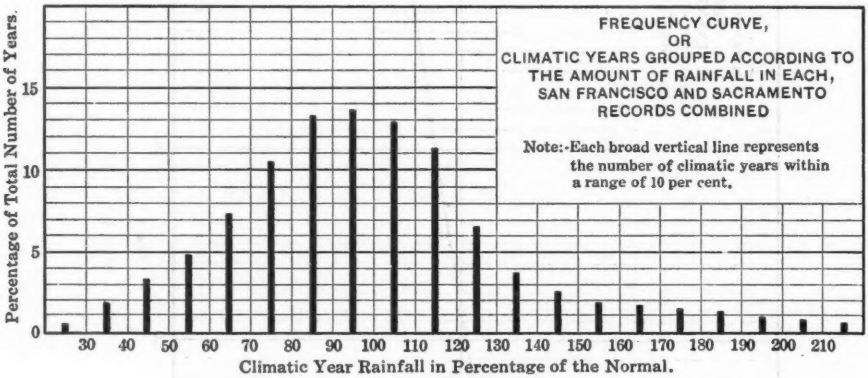

Fig. 5.

of the year the normal seasonal rain to that day in percentage of the annual normal. If, now, the estimated seasonal normal for any station within the region to which a curve of this kind applies, decreases or increases, due to one or more dry or very wet seasons, this change will not materially affect, if at all, the shape of the curve for normal climatic years, but the estimated normal rain to any date will automatically change in the same ratio as the annual normal. A further advantage of using such a mass curve lies in the fact that minor irregularities which appear when the actual mean precipitation to each day of the year is used, are smoothed out, and the true relation of the part-year normal (being the normal to a given date) to the seasonal or full-year normal is more dependably approximated.

The curve, as shown in Fig. 6, being based on the first-class records of rainfall at San Francisco and at Sacramento, is substantially correct for any place in Central California. To illustrate the use of this seasonal mass curve, it may be noted that for February 14th, the normal rainfall applying to that date (being for the season which began on the preceding first day of July) is

* *Journal of Electricity*, Vol. 44, No. 5 (March 1st, 1920).

given by the U. S. Weather Bureau at 14.4 in. for San Francisco and at 12.2 in. for Sacramento. For this date the curve, Fig. 5, shows that 64% of the normal rainfall for a year should have already fallen. For San Francisco, with a normal annual rainfall of 22.7 in., this is 14.5 in. and for Sacramento with a normal annual rainfall of 19.0 in., it is 12.2 in.—values which agree with those determined by the Weather Bureau.

Effect of Altitude on Precipitation.

The attempt is frequently made to establish a relation between the annual rainfall and altitude. A glance at any map showing isohyetose lines will indicate the futility of searching for any such relation that would be widely applicable. It will be found that usually where cyclonic disturbances, accompanied by rain, bring up against, or cross, mountain ranges, the rainfall will be heavier well up on the mountain slope than at the base of the range and that on the

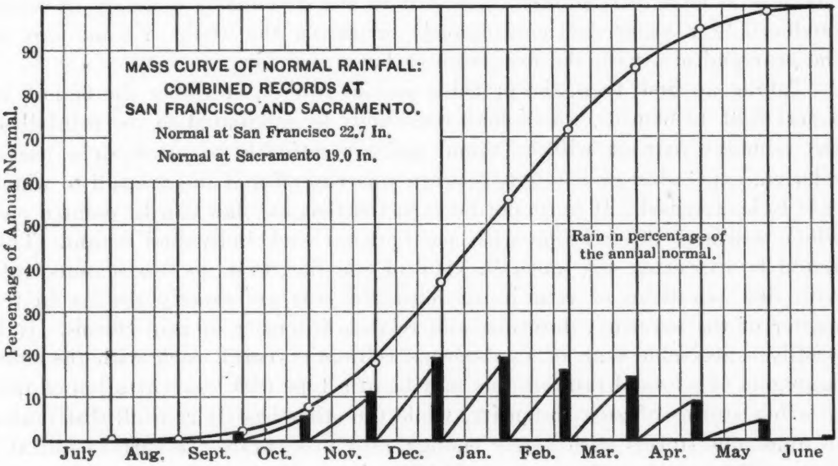

Fig. 6.

far side of the range there will be light rainfall, even though the mountains there may be flanked by a high plateau. The gradient represented by the distance between the isohyetose lines compared with their rain interval, however, may bear almost any kind of a relation to the gradient of the ground surface, and near the summit of the range there is frequently, perhaps it might be said usually, a reversal of the gradient; that is, the region of greatest rainfall is not ordinarily on, but rather somewhat below, the summit of the ridge. It is to be noted, however, that, speaking broadly, the isohyetose lines, or lines of equal quantities of rain, run parallel with the general course of the contour lines of the country. It is, therefore, comparatively easy to extend the isohyetose lines over considerable areas, in regions in which the fall of rain and snow at a few controlling points has been ascertained.

The range in the annual quantities of rain and snow from the western base of the Sierra Nevada, California, across these mountains into Nevada in the latitude of Oroville increases from about 20 in. in the Sacramento Valley to a greatest precipitation of about 80 in. per annum at elevations about 1 000 ft.

below the crest of the range. On the high plateau to the eastward of the main range, the precipitation drops to less than 10 in. per annum, or only about one-eighth as much as at the same elevation on the western slope of the range.

Run-Off in its Relation to Rainfall.

Closely associated with a study of the rainfall in any region is the study of the run-off which results from the rain. The engineer is interested both in the rate at which this run-off occurs and in the quantity thereof in a considerable time-period, as in a year or in a fraction of a year.

In the study of flood-control projects and in the making of provision for adequate storm-water conduits in urban areas, the engineer is particularly concerned with the maximum rate of run-off which the rainfall on a water-shed can produce. In the study of water-power utilization and the making of stream flow available for irrigation and for domestic and industrial use, the engineer is more particularly interested in the dependable quantity of water produced by a water-shed continuously or during the whole or some part of the year and also with the minimum or low-water flow.

Taking up first, then, the probable annual run-off, or rather the run-off in a period of 12 months, which may reasonably be attributed to the rainfall of the climatic year, it will be found quite practicable to establish a useful relation, expressing probability, between this run-off and the rainfall to which it is to be ascribed. It must not be expected that any law can be pointed out, which will give this relation with accuracy for each individual season. This would be expecting too much in view of the fact that no two seasons even with like quantities of rain in the climatic year are exactly similar in the matter of the sequence, duration, and relative intensity of rain storms. It is readily conceivable that, of two seasons (climatic years), each with the same aggregate or seasonal rainfall, one may be of a type with concentration of rain in a few storms of great intensity, while the other has its rainfall distributed to numerous storms with barely enough rain to saturate the surface soil and with sufficient intervals between storms to permit the soil to lose its water by evaporation. In one case, a relatively large proportion of the rain will find its way to the stream; in the other, very little run-off may occur.

The matter is complicated still further in regions where the rainfall, instead of being concentrated in a certain part of the year, is distributed fairly uniformly to the 12 months. In this event, transpiration as well as evaporation may vary within wide limits, particularly in the warm summer months, being affected by the sequence of rains and the great range in the resulting conditions of sunshine, soil moisture, temperature, wind, and other factors which determine the rate at which water, that would otherwise appear in the stream, is carried off by the atmosphere.

Despite any admission that no dependable estimate of run-off can be made for any single year from the known rainfall conditions which cause this run-off, it is, nevertheless, of the utmost importance to determine the ordinary or probable relation between the rain and its effect on the flow of the stream. It is self-evident that no one can predict the rainfall as to time and quantity for future seasons except on the basis of past records. The same amount and

frequency of departures from the normal or probable conditions are to be expected in the future as have been noted in the past. It is only with full appreciation of this limitation on estimated water yield and estimated maximum and minimum flow, that the relation of run-off to rain should be studied. Otherwise, there will be some disappointment and hesitancy in using results, particularly when station records of rainfall are expanded to long time-periods by the plan advocated by the writer. Nevertheless, for such conditions as prevail in the Pacific Coast States, the method is justified and in most cases the results may be accepted as being quite as dependable as though a complete rainfall record were available for each station for which a fragmentary record has been expanded to a full-period record.

The first rain that falls wets the surface of the ground. A continuation of a gentle rain may be at a rate but little if any in excess of that at which the penetration of moisture into the soil takes place. Thus, considerable rain may fall before there is any material run-off. In the interval, thereafter, between one rain storm and the next, the wet ground dries out more or less, its water loss being into the atmosphere by evaporation. If the surface of the ground is kept wet by successive rain storms so that the evaporation, which is greater from a saturated soil than from one that is partly dried out, will be kept at a relatively high rate throughout the time between rain storms, there may be a great loss of water by evaporation. If the rainfall for the climatic year is very light, the time during which the soil is wet and evaporation rapid, will be short; consequently, in such years, evaporation will take less of the rain water than it will in wet years. On the other hand, however, if the number of rainy days is great, and the conditions favoring evaporation, by reason of the long time during which the atmosphere is saturated, are poor, then, even with a saturated condition of surface soils, there may be less evaporation than if a fairly regular alternation of a rainy period of a few days with a dry period of similar duration occurred.

It appears from these considerations that evaporation will take more of the water falling as rain in fairly wet years than in dry years and that evaporation (including transpiration) may take from the soil practically all the water which falls in dry years, when the rainfall is very light, leaving nothing to go to the stream. The probable ratio of evaporation to rainfall will decrease as the seasonal quantity of rain increases up to some more or less indefinite limit and, conversely, the proportion of rain water which reaches the stream as run-off will increase. This fact was recognized by the writer many years ago and led him to formulate for California the following rule, announced in the earlier paper, already referred to, which is applicable in ordinary cases with a fair degree of approximation:

The percentage of the probable run-off due to a rainfall of less than 50 in. in a climatic year of 12 months will be as high as there are inches of rain. The probable run-off due to a rainfall in excess of 50 in. in a climatic year will be 25 in. less than the rainfall; or, expressed by the formula (P = precipitation):

$$r = 0.01\,P^2, \text{ for } P \text{ less than 50 in.} \quad \ldots\ldots(11)$$

$$r = P - 25, \text{ for } P \text{ greater than 50 in.} \quad \ldots\ldots(12)$$

In Equations (11) and (12), P represents the rainfall or precipitation, in inches, in a climatic year, on any water-shed, and r the run-off depth, in inches, over the same area, resulting from this rainfall.

Later studies seem to indicate that quite generally throughout the United States where the precipitation is from 40 to 80 in., the soil and plants growing on the soil will take and dispose of from 22 to 28 in. of water and the remainder will appear in the stream as run-off.

Referring now more particularly to such conditions as prevail in California, the result of the writer's run-off studies early led to the use of two curves, one of which was intended to express the probable relation of run-off to rain in the low or foot-hill regions of the State and the other, showing a somewhat larger proportion of run-off, was intended to show the probable

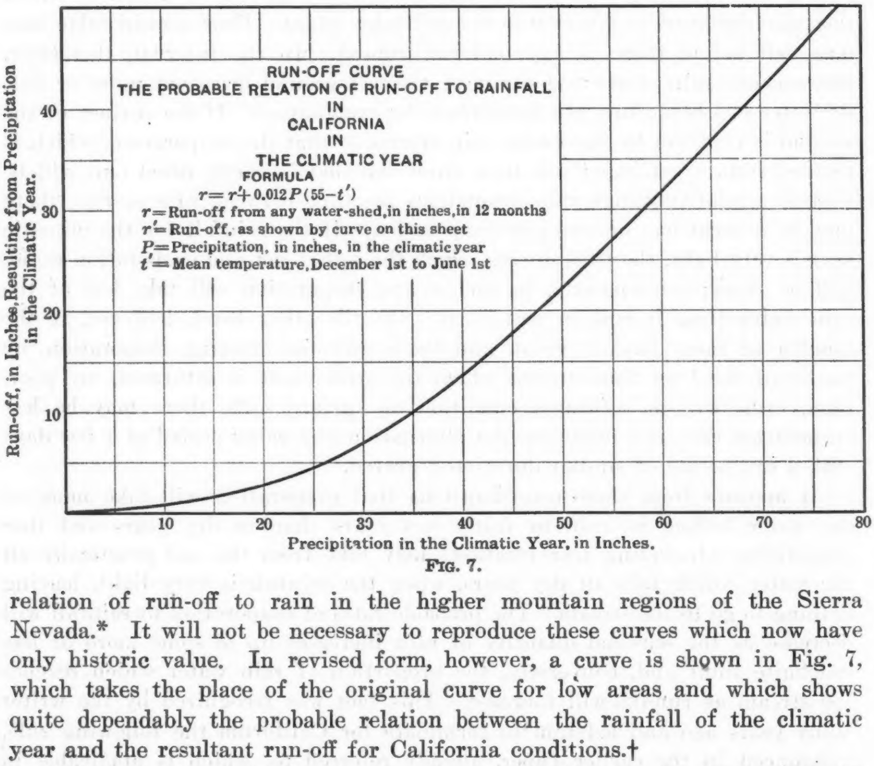

FIG. 7.

relation of run-off to rain in the higher mountain regions of the Sierra Nevada.* It will not be necessary to reproduce these curves which now have only historic value. In revised form, however, a curve is shown in Fig. 7, which takes the place of the original curve for low areas and which shows quite dependably the probable relation between the rainfall of the climatic year and the resultant run-off for California conditions.†

Expressed in tabular form, the writer's run-off curve for water-sheds in California at low altitude gives the values shown in Table 3, which values apply to the run-off resulting from the rainfall in a climatic year of 12 months.

EFFECT OF TEMPERATURE ON RUN-OFF.

It has been found, however, as already suggested that, if applied to high mountain areas, this curve (Fig. 7) would show too little run-off. The cause

* *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 512.

† *Transactions*, Am. Soc. C. E., Vol. LXXIX (1915), p. 166.

TABLE 3.—THE RELATION OF RUN-OFF TO RAINFALL, CALIFORNIA CONDITIONS. (Rain includes snowfall)

Rain, in 12 months of climatic year, in inches.	Resulting run-off, in inches.	Rain, in 12 months of climatic year, in inches.	Resulting run-off, in inches.	Rain, in 12 months of climatic year, in inches.	Resulting run-off, in inches.
5	0.3	24	4.1	55	27.0
10	0.6	26	5.0	60	32.0
12	0.8	28	6.0	65	37.0
14	1.1	30	7.0	70	42.0
16	1.5	35	10.0	75	47.0
18	2.0	40	13.6	80	52.0
20	2.6	45	17.7	90	62.0
22	3.3	50	22.4	100	72.0

for this is not to be sought in the character of the surface of the water-shed, nor yet in the greater altitude of the mountain region, except only as these factors influence temperature and evaporation. All the rain falling on a water-shed, which does not reach the stream, may be considered, as previously stated, to have been returned to the air by evaporation. Some of this rain is returned by transpiration; a small quantity, but generally negligible in its effect on run-off, enters into the composition of tree and plant growth, and a large quantity is lost by evaporation from moist soils, from snowbanks, and from water surfaces of streams and lakes. It is quite apparent that evaporation is the major of these, or the controlling factor; it has seemed proper, therefore, to make an attempt to add a correction increment for temperature or, incidentally for altitude, to the run-off curve, and a suggestion is made in this regard to be tried out as data accumulate.

It is to be assumed, in any event, that evaporation in so far as it materially modifies the run-off from the Sierra Nevada, or, generally, from high mountain areas, as compared with areas at low altitudes, over a period of about six months, December to May, inclusive (California being especially considered), during which snow is on the ground or the ground surface is moist, will be fairly determinable from the temperature of the air.* It may be noted, too, that any such correction for temperature will represent a larger proportion of the run-off when the precipitation is light than when it is heavy. It is true, however, as already noted, that the aggregate evaporation from areas subject to California winter conditions of wetting by snow or rain must be regarded as increasing with increasing precipitation. These considerations have led to the establishment of the correction factor for altitude, or, better, for the lower temperature due to altitude, as appears in the following formula:

Let P = again the rainfall or precipitation, in inches, during the 12 months of the climatic year;

r' = the depth of run-off, in inches, resulting from the precipitation, P, as shown by the curve, Fig. 7, for areas at low altitude;

r = the depth of run-off, in inches, resulting from the precipitation, P, on any area at any altitude; and

* See the writer's discussion of "Evaporation", *Transactions,* Vol. LXXX (1916), p. 1968.

f = the mean temperature, in degrees Fahrenheit, that prevails throughout the water-shed during the period in which evaporation materially affects the run-off. For California, this period will be from December to May.

The formula may now be written for California, as follows:

$$r = r' + 0.012\ P\ (55 - f) \qquad \text{(13)}$$

At a temperature of 55° for the months, December to May, there will be no correction to the values shown by the run-off curve, Fig. 7, at sea level; but, as the elevation increases and the temperature falls, there will be an increment of water, due to less evaporation, to be added to the run-off.

The foregoing formula, Equation (13), was deduced from the more general expression:

$$r = r' + C\,n\,e\,P\ (55 - f) \qquad \text{(14)}$$

in which e represents the increase, in inches, of the monthly evaporation from a water surface, due to an increase of 1° Fahr. in mean monthly temperature (at temperatures of from 40° to 60°); n represents the number of months during which evaporation materially reduces the run-off; and C is a coefficient to be determined by experiment.

For California conditions,* $n = 6$, for the months, December to May, inclusive, $e = 0.1$ in.; and, probably, $C = 0.02$, making $C\,n\,e = 0.012$. To illustrate the use of the curve and formula for the run-off in a 12-month period, the case of a Sierra Nevada mountain area, in California, at an altitude of 6 000 ft., may be taken, on which the climatic year shows a rainfall of 40 in.

From the curve, Fig. 7, the value of r' for 40 in. of rain appears at 13.6 in. In the Sierra Nevada, at altitudes of 6 000 ft., the mean temperature for the six months, December to May, inclusive, is about 40 degrees. Consequently, the seasonal run-off by formula is as follows:

$$r = 13.6 + 0.012 \times 40\ (55 - 40)$$
$$r = 13.6 + 7.2 = 20.8 \text{ in.}$$

It will be noted from this illustration that, due to low temperature at high altitudes, there may be a material increase in the proportion of rain which finds its way to the stream.

Normal Run-Off Computed from Rainfall Records.

After the relation between seasonal (12 months) rainfall and probable run-off has been established by some such curve as that shown in Fig. 7, in figures such as those presented in Table 3, or by the formula, Equation (13), it becomes possible to compute the probable or normal run-off for each 12-month period, for any region, from the rainfall record, when the rainfall is expressed for each season in percentage of the normal rainfall. For all practical purposes and particularly as a means of forecasting future run-off, this method may be accepted with confidence. It leads to a computed record of stream flow which will show what the water output of the stream has been and what water output is to be expected in the future. It shows, too, the relation of the

* *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1968.

water output in minimum and maximum years to the normal water production and also the frequency of years of ordinary, small, and large water production.

As an illustration, a stream may be selected on the water-shed of which the mean annual rainfall has been found to be 30 in. and the seasonal variation of which in the quantity of rain is represented by the composite rain record noted in Table 2.

It follows that in this water-shed, in a series of 100 years, rainfall and run-off will occur about as set forth in Table 4.

TABLE 4.—COMPUTATION OF NORMAL ANNUAL RUN-OFF FOR A NORMAL SEASONAL RAINFALL OF 30 IN. FROM A WATER-SHED AT LOW ALTITUDE IN A REGION FOR WHICH THE COMPOSITE RECORD OF RAINFALL AT SAN FRANCISCO AND SACRAMENTO IS TYPICAL.

Number of climatic years.	Rainfall in percentage of normal.	Annual rainfall, in inches.	Resulting annual run-off, in inches.	Total run-off for period, in inches.
0.5	20	6.0	0.40	0.20
1.7	35	10.5	0.65	1.11
3.3	45	14.5	1.20	3.96
4.9	55	16.5	1.63	7.99
7.3	65	19.5	2.45	17.89
10.4	75	22.5	3.50	36.40
13.1	85	25.5	4.77	62.49
13.6	95	28.5	6.25	85.00
12.9	105	31.5	7.90	101.91
11.3	115	34.5	9.70	109.61
6.6	125	37.5	11.80	77.88
3.7	135	40.5	14.00	51.80
2.6	145	43.5	16.50	42.90
1.9	155	46.5	19.10	36.29
1.6	165	49.5	21.90	35.04
1.3	175	52.5	24.70	32.11
1.9	190	57.0	29.00	58.10
1.4	205	61.5	33.50	46.90
100.	......	......	......	804.58
Mean......	100	30.0	......	8.0

According to the calculation presented in Table 4, the normal run-off for a normal seasonal rainfall of 30 in. (that is, within a climatic year of 12 months), is 8.0 in. from a water-shed at low altitude in central portions of California. The probable run-off, on the other hand, resulting from a like quantity of rain, that is, 30 in., during any single climatic year, according to the curve, Fig. 7, or Table 3, is 7.0 in. The normal annual run-off, therefore, from a water-shed at low altitude, throughout which the normal annual rainfall is 30 in., exceeds the probable run-off, due to a like quantity of rain in the single climatic year, by 1 in., or by about 14 per cent. A similar calculation of the normal run-off when the normal rainfall is 20 in. will show an excess over the probable single season run-off, for a like quantity of rain, of about 19 per cent. For a normal seasonal rainfall of 40 in., the excess is 9%; for a normal seasonal rainfall of 50 in., it is 3%; and for a normal seasonal rainfall of 60 in., the normal annual run-off will be about 2% less

than that which is probable in any single climatic year with a rainfall of 60 in.

These relations apply, as noted, to a region in which rainfall conditions are correctly represented by the composite rainfall records, as shown in Fig. 3, applying to certain central areas of California. Where the variation in the rainfall in the several climatic years of a long time-period departs from that shown by this diagram, a different relation between the normal seasonal run-off and the estimated or probable run-off in a season with a rainfall equal to the normal may be found. The numerical illustration is intended merely as a guide in making run-off studies. A different relation will be found, too, when correction factors are applied for altitude or temperature departures from those to which the run-off curve, Fig. 7, applies.

When the run-off is to be determined from a water-shed of so great an extent that there is a wide range in the normal rainfall on different portions thereof, it should be subdivided, preferably in such fashion that the range of normal rainfall in each subdivision will not exceed 5 in., and the estimate of the year's run-off from each subdivision can then be made separately.

The isohyetose lines shown in Fig. 8, for a low to high mountain region lying to the eastward of Oroville, Cal., may serve to illustrate not only the considerable variation of normal annual rainfall within comparatively short distances, but also the difficulty that would exist, in such an area as that covered by the diagram, of estimating the average normal annual rainfall for the entire area from a few isolated station records without recourse to isohyetose lines.

Elements of Uncertainty.

In concluding this discussion of the relation of run-off to rainfall, it may be noted that the problem is complicated by the difficulty always encountered of determining the precipitation on any area of considerable extent, as well as by the lack of precision in the estimates of stream flow. The crudest kind of approximation must usually be resorted to in the matter of rainfall. When water-sheds, for which the shape of the isohyetose lines is known, are studied in relation to rainfall records at a few individual points, the opportunity for error in this particular becomes clearly apparent. In the case of drainage areas such as those in the high mountains of California, for example, there is much uncertainty, too, as to the normal annual snowfall and the snowfall in any individual season. Even for any single point this statement holds good. The distribution of snow over considerable areas with all kinds of exposure to the wind cannot be uniform, and there is uncertainty, too, as to the quantity of water which any given depth of snowfall may represent. The few records of precipitation in California's snowbelt, which are available, are, at best, to be accepted as approximate only, and, yet, it appears that, generally, in respect to precipitation distribution, the information available in this State is as reliable as any elsewhere in this country.

Maximum Storm-Water Flow from Small Areas.

A complete discussion by the writer of the storm-water flow problem as applied to urban areas will be found in his paper on "The Sewer System

of San Francisco."* The value of the deduced formulas, as there presented, may be reaffirmed, and a wider application may be claimed for them, provided only that fair information is obtainable from past records of rain intensities throughout the affected areas. The element of greatest uncertainty in any formula for the maximum run-off rate will always be the perviousness of the surface of the water-shed. In each case allowance for the effect of this perviousness must be made by the engineer, on the basis of experience.

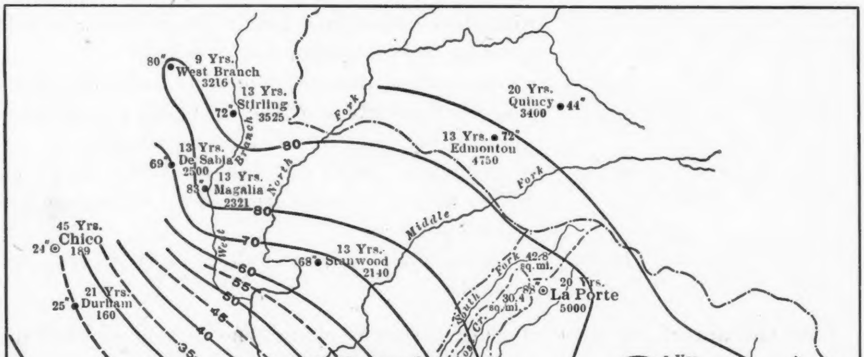

Fig. 8.

Let A_1, A_2, A_3, etc., = the areas of subdivisions of any water-shed, in acres;

A = the area, in acres, of the water-shed;

a, a_1, a_2, a_3, etc., = the coefficients of perviousness, or run-off coefficients, applying to the water-shed and the respective subdivisions thereof;

S = the average slope of the main watercourse, in feet per thousand;

* *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 310.

T = the time, in minutes, that it will take water to flow, under maximum run-off conditions, in natural or in proposed conduits, from the most remote part of the water-shed to the point at which the maximum storm-water discharge is to be determined;

t = the critical time, in minutes, during the continuance of a rain-storm for the area under consideration, being that time within which the rain will produce the maximum rate of run-off.

i = the number of minutes which it takes water to flow on the surface of the area under consideration to the points of entry into the conduits;

d_m = the maximum storm-water flow at the point at which flow is to be determined, in cubic feet per second per acre;

D_m = the maximum storm-water flow, in cubic feet per second.

By the aid of the rain-intensity formulas already noted and of observations throughout the water-shed, which establish the maximum quantity of rain which may be expected to fall thereon in periods of 30 min., 1 hour, 2 hours, or some other period, or periods, of time, the value of the intensity coefficient, C, in the formula for I, Equation (1), namely,

$$I = \frac{C}{\sqrt{t}}$$

is to be determined, and the procedure when the capacity to be given to storm-water conduits is in question, will then be as follows:

Divide the water-shed having an area of A acres into three subdivisions, A_1, A_2, A_3, such that the time required by water to flow from the farther limits of the first over the surface and in conduits of the type which are to be provided, will be one-third, and that the time required for water to flow from the most remote parts of the second subdivision, will be two-thirds of the time required for storm-water to flow from the most remote part of the water-shed to the point at which maximum storm-water flow is to be estimated.

Approximate the time, i, in minutes, that it will take water to flow on the surface of the water-shed to the inlets of the conduit system. Estimate the time, T, in minutes, that it will take water to flow from the most remote part of the water-shed to the point at which maximum flow is to be estimated. Then, determine the critical time, t, from the following:

$$t = 0.40 \frac{T}{a\,A} (a_1\,A_1 + 2\,a_2\,A_2 + 3\,a_3\,A_3) + i \quad \ldots\ldots\ldots\ldots (15)$$

The coefficient of imperviousness, a, for the entire water-shed is found from the following:

$$a = \frac{a_1\,A_1 + a_2\,A_2 + a_3\,A_3}{A} \quad \ldots\ldots\ldots\ldots\ldots\ldots (16)$$

Consideration must be given to the perviousness of the surface of the watershed in determining the values of a_1, a_2, and a_3. For an entirely impervious surface and relatively small areas, these factors would be unity; for a coarse, well under-drained gravel, they may approximate zero. The values of a, a_1, a_2, and a_3, therefore, will lie somewhere between unity and zero.

For use in a similar formula for maximum flow, the late Emil Kuichling, M. Am. Soc. C. E., based on large experience, recommended* the following:

	Value of a.
For roof surfaces	0.70 to 0.95
" asphalt pavements in good order	0.85 to 0.90
" stone, brick, and wooden block pavements with tightly cemented joints	0.75 to 0.85
" same, with open or uncemented joints	0.50 to 0.70
" inferior block pavements with uncemented joints	0.40 to 0.50
" macadamized roadways	0.25 to 0.50
" gravel roadways and walks	0.15 to 0.30
" unpaved surfaces, railroad yards, and vacant lots	0.10 to 0.30
" parks, gardens, lawns, and meadows, depending on surface slope and character of subsoil	0.05 to 0.25
" wooded areas or forest land, depending on surface slope and character of subsoil	0.01 to 0.20

It is believed, in applying the formulas, which are here presented, that it would be better not to attempt the minute classification of surface according to its character, which was suggested by Mr. Kuichling. When small urban areas are under consideration, all surfaces which approximate imperviousness may well be put into one class, because the infiltration of water into the surface material may be neglected when the critical time is short and the rain intensity high.

When larger areas are under consideration, and the soil of the country is an ordinary loam, the approximate values given in Table 5, based on population density, may be used.

TABLE 5.

Population per acre.	Value of a.	Population per acre.	Value of a.
10	0.20	60	0.60
20	0.30	70	0.65
30	0.40	80	0.70
40	0.50	90	0.73
50	0.55	100	0.75

The maximum storm-water flow can now be estimated from the value of I, or from the maximum rainfall, R, in 1 hour, by the formula:

$$d_m = 0.645\ a\ I \text{ sec-ft per acre} \qquad (17)$$

making,

$$D_m = 0.645\ a\ A\ I \text{ sec-ft} \qquad (18)$$

* *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 399.

or its equivalent,

$$d_m = \frac{5\,a\,R}{\sqrt{t}} \text{ sec-ft. per acre} \ldots\ldots\ldots\ldots (19)$$

making,

$$D_m = \frac{5\,a\,A\,R}{\sqrt{t}} \text{ sec-ft.} \ldots\ldots\ldots\ldots (20)$$

In developing this formula, consideration has been given to the effect of the temporary water storage (that is, to the increase or decrease of the volume of water in transit) in the conduits as well as that resting on the surface of the water-shed, on the momentary rate of flow. This is fully explained in the writer's paper* to which reference has already been made.

The value of I is best obtainable from the diagram, Fig. 3, or by Equations (1) to (5). The value of C in the rain-intensity formula, Equation (1), as explained, is dependent on local rainfall conditions. Local records of rainfall, or if no local records are available, then those of some locality with similar meteorological conditions, are to be used in determining the value of C and the resultant rain intensities.

When water-sheds are under consideration, which, though still belonging in the class of small areas, are of so large an extent and so diversified in surface characteristics that the maximum rain intensities in different portions thereof have a wide range, that is, depart widely from the values that would be most applicable for the water-shed treated as a unit, there should be introduced into the calculation the maximum rainfall per hour applying to each subdivision thereof, R_1, R_2, R_3, or the corresponding coefficients, C_1, C_2, C_3, but these should always be determined for the critical time, t, which applies to the entire water-shed.

The formulas may then be written:

$$D_m = \frac{5}{\sqrt{t}}\,(a_1\,A_1\,R_1 + a_2\,A_2\,R_2 + a_3\,A_3\,R_3) \ldots\ldots\ldots\ldots (21)$$

or,

$$D_m = 0.645\,(a_1\,A_1\,I_1 + a_2\,A_2\,I_2 + a_3\,A_3\,I_3) \ldots\ldots\ldots\ldots (22)$$

The formulas previously noted for maximum run-off, are simple in form and convenient for general use. The numerical factors appearing therein have been deduced in part, as previously stated, from a study of the effect of the constantly changing volume of water actually in transit over the surface and in the conduits, that is, of the effect (on the flow) at the gauging point of an increase or decrease of the quantity of water temporarily in storage within the water-shed. The factors which will always be involved in some uncertainty, are the coefficients of perviousness and of rain intensity.

The following example (based on assumed data) will illustrate the application of the formula for maximum stream flow.

* *Transactions*, Am. Soc. C. E., Vol. LXV (1909), p. 294.

A topographical survey and preliminary estimates of the velocity at which water will flow in proposed conduits, have established the following values:

$$
\begin{aligned}
A &= 2\,500 \text{ acres} \\
A_1 &= 500 \text{ acres}; \; a_1 = 0.90 \\
A_2 &= 1\,200 \text{ acres}; \; a_2 = 0.75 \\
A_3 &= 800 \text{ acres}; \; a_3 = 0.40 \\
T &= 85 \text{ min.} \\
i &= 10 \text{ min.}
\end{aligned}
$$

The available rainfall records show, for this water-shed, that during one storm 1 in. of rain fell in 2 hours and, at another time, it rained 2 in. in 6 hours. It is known, too, that the storms in which rainfalls of this intensity occurred, were classed as severe, and that both records may be accepted as approaching single-station maxima. Being for application to a fairly large urban area, the values of R and C deduced from these records will probably be somewhat in excess of the probable value for the whole area, and this excess may be considered as some margin of safety, although not quite enough.

The rain intensity of 0.50 in. per hour deduced from the 2-hour record indicates a value of $C = 5.5$; and the rain intensity of 0.33 in. per hour from the 6-hour record indicates a value of $C = 6.2$.

As the extreme rain intensity may exceed that of either of the only two storms for which dependable records exist, it will be proper to introduce C into the calculation at a value of about 6.5.

Then, by Equation (12):

$$a = \frac{0.90 \times 500 + 0.75 \times 1\ 200 + 0.40 \times 800}{2\ 500} = 0.66$$

by Equation (11):

$$t = 0.40 \left(\frac{85}{0.66 \times 2\ 500}\right) (0.90 \times 500 + 2 \times 0.75 \times 1\ 200 + 3 \times 0.40 \times 800) + 10 = 76 \text{ min.}$$

by Equation (1)

$$I = \frac{6.5}{\sqrt{76}} = 0.74$$

and by Equation (15):

$$d_m = 0.645 \times 0.66 \times 0.74 = 0.32 \text{ cu. ft. per sec. per acre}$$

and by Equation (16):

$$D_m = 0.32 \times 2\ 500 = 800 \text{ cu. ft. per sec.}$$

This is the estimated maximum storm-water flow from the 2 500 acres.

Maximum Stream Flow or Maximum Rate of Run-Off from Large Areas.

When the capacity is to be prescribed of a spillway for a storage reservoir, or of a stretch of river to which flood-waters are to be confined, larger areas come under consideration than in the case of ordinary urban problems. It is

then desirable to express areas in square miles instead of in acres. With this change of area unit, but again under the assumption that meteorological conditions and, therefore, values of R, the maximum rainfall in 1 hour, and of I, the maximum average rate of rainfall in the critical time-period, t, may be considered to be uniform throughout the water-shed, there will be:

From Equation (17):

$$d'_m = 413\ a\ I \text{ sec-ft. per sq. mile} \quad \text{(23)}$$

from Equation (19):

$$d'_m = \frac{3\ 200\ a\ R}{\sqrt{t}} \text{ sec-ft. per sq. mile} \quad \text{(24)}$$

from which,

$$D_m = 413\ a\ M\ I \text{ sec-ft.} \quad \text{(25)}$$

or,

$$D_m = \frac{3\ 200\ a\ M\ R}{\sqrt{t}} \text{ sec-ft.} \quad \text{(26)}$$

in which M represents the area of the water-shed, in square miles.

In applying this formula to large areas, it must be remembered that the individual station records of rainfall do not represent intensities as dependably for large areas as they do for small areas. Whenever practicable the relation of the rainfall on the entire water-shed to that of the single station or, better, to the combined records at a number of stations, should be ascertained. This is done by comparing the station records with the rainfall on the whole area, as determined by isohyetose lines.

The factor, R, in the writer's formulas for the maximum rate of run-off, should always represent an average value for the area to which it applies. That is, if the maximum rain to be expected in one hour has been determined for numerous places regularly distributed throughout the area, the value of R to be used in the formula will be the mean of all such determinations.

When there is a wide range in the meteorological conditions in the subdivisions of a water-shed (which need not be restricted to only three), and there is a wide range in the values of R and of I in different parts thereof, it may be advisable to use the formulas in a more general form, deduced from Equations (21) and (22). They can then be written:

$$D_m = \frac{3\ 200}{\sqrt{t}} (a_1\ M_1\ R_1 + a_2\ M_2\ R_2 + a_3\ M_3\ R_3) \text{ sec-ft.} \quad \text{(27)}$$

or,

$$D_m = 413\ (a_1\ M_1\ I_1 + a_2\ M_2\ I_2 + a_3\ M_3\ I_3) \text{ sec-ft.} \quad \text{(28)}$$

in which, as explained, $M_1 + M_2 + M_3 = M$, and all these areas are expressed in square miles.

Due to the fact that the absorption of water by soil or other pervious material, as well as the rate of evaporation, may be regarded as fairly constant, while the average intensity of the rainfall decreases as the time to which the intensity applies increases, the relative effect of perviousness will increase in some measure as the critical time increases. This statement has, of course,

particular application to large water-sheds in which the critical time may be measured by days with occasional cessation of rainfall instead of by minutes and hours, as is the case in urban problems. The coefficient of perviousness, that is, a in the formula for urban areas would better be considered as a run-off coefficient and should decrease as the critical time increases. Bearing in mind that $a = 1$ for small impervious areas, the following expression for a has been found to give results in fair conformity with observed facts:

$$a = \frac{60}{60 + c\sqrt[3]{t}} \quad \text{.......................(29)}$$

Here, c may be regarded as a supplemental run-off coefficient with a wide range in value, being almost negligible for impervious areas and increasing with increasing perviousness. A value should be assigned to c with due regard to surface conditions of the water-shed. The following values of c are tentatively advanced for large drainage areas:

For impervious areas........................ $c = 0.5$
For mountainous areas....................... $c = 5.$
For low rolling country..................... $c = 20.0$
For flat country (ordinary soil)............... $c = 50.$
For sandy regions........................... $c = 250.$

These values of c, intended to apply to conditions as they ordinarily obtain in temperate climates, may be found to be too large, giving maximum run-off rates which are too small in localities where the ground may be frozen, or water-logged, or where the maximum run-off rate occurs when rain falls on snow.

In Table 6 some values of the run-off coefficient, a, for various values of c, are given.

TABLE 6.—VALUES OF THE RUN-OFF COEFFICIENT.

Based on: $a = \dfrac{60}{60 + c\sqrt[3]{t}}$

Time in, minutes.	Impervious areas, $c = 0.5$.	Mountain areas * $c = 5$.	Rolling country,* $c = 20$.	Flat country, $c = 50$.	Sandy regions, $c = 250$
1	0.99	0.92	0.75	0.55	0.19
8	0.98	0.86	0.60	0.37	0.107
27	0.98	0.80	0.50	0.29	0.074
64	0.97	0.75	0.43	0.23	0.057
125	0.96	0.71	0.37	0.19	0.046
216	0.95	0.66	0.33	0.17	0.039
343	0.94	0.63	0.30	0.15	0.033
512	0.94	0.60	0.27	0.13	0.029
729	0.93	0.57	0.25	0.118	0.026
1 000	0.92	0.55	0.23	0.107	0.023
1 321	0.91	0.52	0.21	0.098	0.021
1 728	0.91	0.50	0.20	0.091	0.019
2 197	0.90	0.48	0.19	0.084	0.018
2 744	0.90	0.46	0.18	0.079	0.017
3 375	0.89	0.44	0.17	0.074	0.016
8 000	0.86	0.37	0.13	0.057	0.012
15 625	0.83	0.32	0.11	0.046	0.010
27 000	0.80	0.29	0.091	0.038	0.008

*Mountain areas are assumed to have considerable rock surface, or rock thinly covered with soil, and rolling country some parts that are considerably less pervious than ordinary soil.

By inserting in Equations (23) to (26), the value of a, Equation (29), and using round numbers, the formulas for maximum run-off are as follows:

$$d'_m = \frac{25\ 000\ I}{60 + c\ t^{0.33}} \text{ sec-ft. per sq. mile} \qquad (30)$$

or,

$$d'_m = \frac{190\ 000\ R}{60\ t^{0.5} + c\ t^{0.83}} \text{ sec-ft. per sq. mile} \qquad (31)$$

and,

$$D_m = \frac{25\ 000\ M\ I}{60 + c\ t^{0.33}} \text{ sec-ft.} \qquad (32)$$

or,

$$D_m = \frac{190\ 000\ M\ R}{60\ t^{0.5} + c\ t^{0.83}} \text{ sec-ft.} \qquad (33)$$

The use of the formula is made convenient by Table 7 and by a diagram, Plate V. In both Table 7 and Plate V, the value of the factor is shown which, when multiplied by the maximum rainfall in 1 hour, R, expressed in inches, will give the maximum run-off rate, in second-feet per square mile.

The value of t is determinable, as has been explained, from the topographic features of the water-shed and the character of the storm-water conduits. The value of c will be adopted from known characteristics of the ground surface, with special regard to perviousness.

The formula is based on the assumption that $I = \frac{C}{\sqrt{t}}$ (Equation (1)). The appearance of I in the formula is not necessary when R appears therein.

It will be seen by inspection of Plate V, that straight lines substituted for the curves there shown would afford a fairly good approximation to the values given by the formula. A coefficient, which may be called C'', and the exponent of t, will then be subject to selection according to the surface characteristics of the water-shed. The approximation formula for maximum run-off, in second-feet per square mile, may then be written:

$$d'_m = \frac{C''\ R}{t^x} \text{ sec-ft. per sq. mile} \qquad (34)$$

where,

$C'' = 3\ 500$ and $x = 0.5$ for impervious areas;
$C'' = 3\ 300$ and $x = 0.6$ for mountainous areas;
$C'' = 3\ 000$ and $x = 0.7$ for rolling country;
$C'' = 2\ 100$ and $x = 0.75$ for flat country; and
$C'' = 600$ and $x = 0.8$ for sandy regions.

Whenever large areas are under consideration, that is, when maximum stream or river flow, as distinguished from the storm-water flow of urban conduits, is to be determined, the value of R should be estimated from the maximum rainfall in periods of 24 hours or more, rather than from the maximum rainfall in shorter periods.

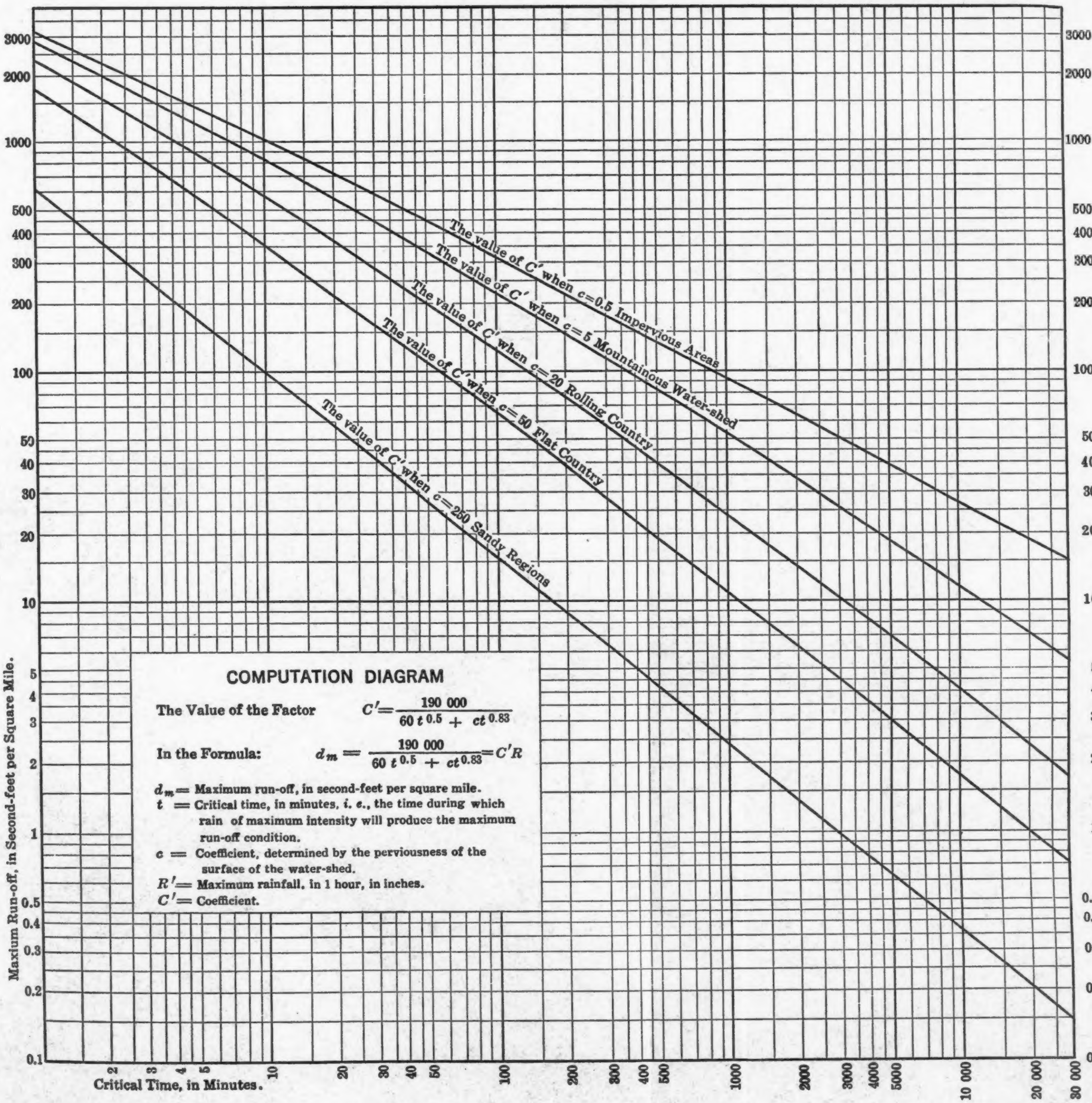
Maxium Run-off, in Second-feet per Square Mile.
Critical Time, in Minutes.
The value of C′ when c=0.5 Impervious Areas
The value of C′ when c=5 Mountainous Water-shed
The value of C′ when c=20 Rolling Country
The value of C′ when c=50 Flat Country
The value of C′ when c=250 Sandy Regions
COMPUTATION DIAGRAM
The Value of the Factor $C' = \frac{190\ 000}{60\,t^{0.5} + ct^{0.83}}$
In the Formula: $d_m = \frac{190\ 000}{60\,t^{0.5} + ct^{0.83}} = C'R$
d_m = Maximum run-off, in second-feet per square mile.
t = Critical time, in minutes, *i. e.*, the time during which rain of maximum intensity will produce the maximum run-off condition.
c = Coefficient, determined by the perviousness of the surface of the water-shed.
R' = Maximum rainfall, in 1 hour, in inches.
C' = Coefficient.

TABLE 7.—THE VALUE OF THE FACTOR, $C' = \dfrac{190\ 000}{60\ t^{0.5} + c\ t^{0.83}}$, FOR VARIOUS VALUES OF t AND c, IN THE FORMULA, $d'_m = \dfrac{190\ 000\ R}{60\ t^{0.5} + c\ t^{0.83}} = C'\ R$.

Critical time, t, in minutes.	$c = 0.5$, impervious area, in second-feet.	$c = 5$, mountains, in second-feet.	$c = 20$, rolling country, in second-feet.	$c = 50$, flat country, in second-feet.	$c = 250$, sandy regions, in second-feet.
1	3 140	2 920.0	2 380.00	1 730.00	610.00
8	1 100	960.0	671.00	425.00	120.00
27	595	488.0	304.00	174.00	45.00
64	383	297.0	170.00	91.00	20.50
125	272	199.0	106.00	55.00	12.10
216	205	144.0	72.00	36.00	7.80
348	161	108.0	51.00	25.00	5.70
512	131	84.0	38.00	18.20	4.10
729	109	67.0	29.00	13.80	2.90
1 000	92	55.0	23.00	10.70	2.30
1 321	79	45.0	18.70	8.60	1.86
1 728	69	38.0	15.20	6.90	1.49
2 197	61	32.0	12.70	5.70	1.22
2 744	54	28.0	10.70	4.80	1.02
3 375	48	24.0	9.10	4.00	0.86
8 000	30	13.3	4.60	2.00	0.42
15 625	21	8.2	2.70	1.16	0.24
27 000	15	5.5	1.75	0.74	0.15

NOTE.—To find the maximum run-off per square mile in second-feet multiply the figures in Table 7, by the value of R, that is, the maximum rainfall in 1 hour expressed in inches.

To show the effect of area on the maximum stream flow or maximum rate of run-off, areas of similar outline and similar topographical features and subject to the same meteorological conditions may be compared with each other.

Because areas of various extent, but similar in topography and outline, will have values of t fairly proportional to the square root of the surface area, it may be assumed for such areas that,

$$t = K \sqrt{M} \qquad (35)$$

in which K is a coefficient that can be determined for any set of water-sheds complying with the condition of similarity.

Let it now be assumed that for a number of mountain water-sheds of similar characteristics, a value of $K = 20$ has been ascertained. This will make for this special case:

$$t'' = 20 \sqrt{M}$$

and from Equation (23),

$$d''_m = \frac{715\ a\ R}{(M)^{\frac{1}{4}}}.$$

Mountainous regions being under consideration, there will be:

$$a = \frac{60}{60 + 5 \sqrt[3]{t}}$$

Therefore, inserting the value of t'',

$$a = \frac{60}{60 + 14\ M^{0.17}}$$

for the special case only.

The following values, Table 8, result for this special case when water-sheds range in area from 10 to 10 000 sq. miles.

TABLE 8.—RELATIVE MAXIMUM STREAM FLOW FROM SIMILAR MOUNTAIN WATER-SHEDS OF VARIOUS EXTENT.

In this Table, R Represents Maximum Rainfall in One Hour. (Special Case, only, $K = 20$)

Area, in square miles.	Critical time, in minutes.	Maximum flow, in second-feet per square mile.	Total maximum flow, in second-feet.
10	63	302 R	3 000 R
20	89	250 R	4 880 R
50	142	188 R	9 440 R
100	200	149 R	14 700 R
200	283	122 R	24 400 R
300	346	108 R	32 400 R
400	400	99 R	39 600 R
500	450	92 R	46 000 R
1 000	630	75 R	74 000 R
5 000	1 410	44 R	220 000 R
10 000	2 000	34 R	340 000 R

The values in Table 8 show that in all mountainous regions exposed to the same rainfall conditions, the rate of maximum run-off, as determined by the formula, is about nine times greater per square mile from a water-shed having an area of 10 sq. miles and about four to five times greater from 100 sq. miles than from a water-shed having an area of 10 000 sq. miles. In the light of the stream-flow information obtainable from records of the U. S. Geological Survey, and other sources, and the conclusions reached in this matter by other investigators (Fig. 9), this relation appears to be quite reasonable. When account is taken of the fact that the value of R, that is, the maximum rainfall in 1 hour, may be much larger for those spots within the larger area at which the rain storms break with greatest severity, it will be readily seen that the disparities between the maximum run-off rate from the small area and that from the large area, as noted, which are predicated on the same value of R throughout the small area as throughout the larger area, may be considerably exceeded.

Other formulas for the approximation of maximum run-off under extreme conditions of rainfall are numerous. Most of these are intended to serve in restricted territory, throughout which a similarity of rainfall conditions permits the use of run-off coefficients which are practically independent of the variations in the intensity of rainfall. Of this type, are the formulas suggested by the late Mr. Kuichling* for the New England and North Atlantic States; the Metcalf and Eddy formula;† the formula used by W. E. Fuller,‡ M. Am. Soc. C. E., and many others. Such formulas will naturally be used with some hesitation until they are supplemented with suitable correction factors to adapt them to meteorological conditions which depart materially

* "Report on the Barge Canal of New York," 1901, Part 14 of the "Report on Water Supply," by Emil Kuichling, p. 844.

† "American Sewerage Practice," by Metcalf and Eddy, p. 251.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVII (1914), p. 564.

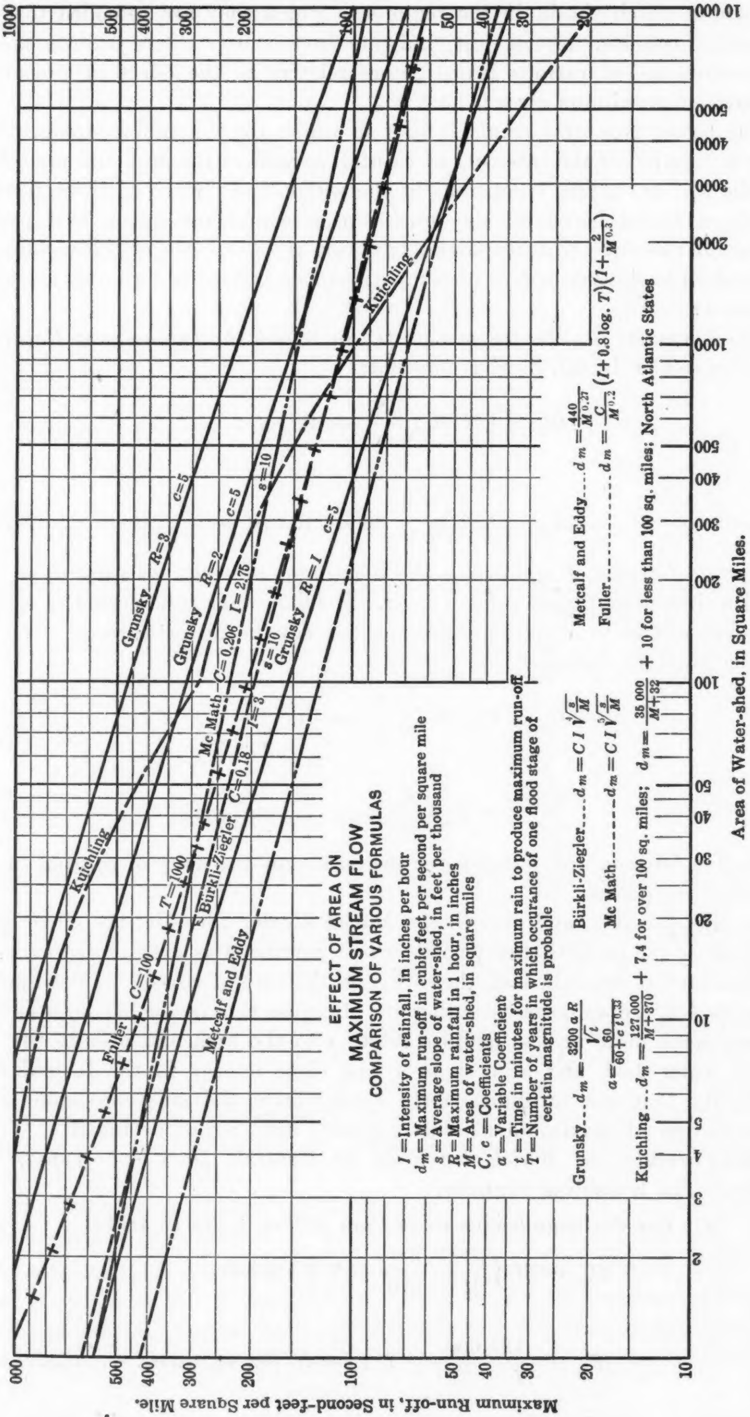
EFFECT OF AREA ON
MAXIMUM STREAM FLOW
COMPARISON OF VARIOUS FORMULAS
I = Intensity of rainfall, in inches per hour
d_m = Maximum run-off in cubic feet per second per square mile
s = Average slope of water-shed, in feet per thousand
R = Maximum rainfall in 1 hour, in inches
M = Area of water-shed, in square miles
C, c = Coefficients
a = Variable Coefficient
t = Time in minutes for maximum rain to produce maximum run-off
T = Number of years in which occurance of one flood stage of certain magnitude is probable
Grunsky... $d_m = \frac{3200\,a\,R}{\sqrt{t}}$ $a = \frac{60}{60 + c\,t^{0.33}}$
Bürkli-Ziegler..... $d_m = C\,I\,\sqrt[4]{\frac{s}{M}}$
Mc Math........ $d_m = C\,I\,\sqrt[5]{\frac{s}{M}}$
Metcalf and Eddy..... $d_m = \frac{440}{M^{0.27}}$
Fuller........ $d_m = \frac{C}{M^{0.2}}(I + 0.8 \log. T)\left(I + \frac{2}{M^{0.3}}\right)$
Kuichling ... $d_m = \frac{127\,000}{M+370} + 7.4$ for over 100 sq. miles; $d_m = \frac{35\,000}{M+32} + 10$ for less than 100 sq. miles; North Atlantic States
Grunsky R=3 c=5
Grunsky R=2 c=5
Grunsky R=1 c=5
Mc Math C=0.206 I=2.75 s=10
Bürkli-Ziegler C=0.18 I=3 s=10
Fuller C=100 T=1000
Kuichling
Metcalf and Eddy
Maximum Run-off, in Second-feet per Square Mile.
Area of Water-shed, in Square Miles.
1000 500 400 300 200 100 50 40 30 20 10
2 3 4 5 10 20 30 40 50 100 200 300 400 500 1000 2000 3000 4000 5000 10 000

FIG. 9.

from those which obtain in the regions to which they are intended to apply. All such formulas, nevertheless, indicate in a more or less definite manner the conclusions of various investigators relative to the effect of topography and area on maximum stream flow.

The better type of formula is that in which the maximum stream flow is made a function of the intensity of rainfall as well as the area and topography and the soil or surface conditions of the water-shed. Most of these formulas contain a factor based on the precipitation conditions which will produce maximum run-off. Unfortunately, however, it is not always clear how this factor shall be determined. For comparison, the following formulas are briefly discussed:

A.—Formulas for Maximum Run-Off in Which the Intensity of Rainfall is a Factor.—The Bürkli-Ziegler formula:

$$D_m = C\,I\,M \sqrt[4]{\frac{S}{M}} \text{ sec-ft.} \quad \text{.........................(36)}$$

and,

$$d_m = C\,I \sqrt[4]{\frac{S}{M}} \text{ sec-ft. per sq. mile} \quad \text{................(37)}$$

In this formula, I stands for the maximum intensity of rainfall in some definite time-period, preferably 1 hour. If some other time-period is selected, the effect will be to alter the coefficient, but not the ultimate result.

The McMath formula:

$$D_m = C\,I\,M \sqrt[5]{\frac{S}{M}} \text{ sec-ft.} \quad \text{.........................(38)}$$

and

$$d_m = C\,I \sqrt[5]{\frac{S}{M}} \text{ sec-ft. per sq. mile} \quad \text{................(39)}$$

In this formula, too, I stands for the maximum intensity of rainfall in some definite time-period.

In this formula, and also in the Bürkli-Ziegler formula, the effect of the duration of the critical time-period, which lengthens as area increases, on the average rate of rainfall, and, therefore, on the rate of run-off, is not apparent as the factor, t, does not appear therein. Because this factor has been assumed to bear some more or less definite relation to the area and also to the slope of the water-shed, and as both area and slope appear in the formulas, the possibility that the formulas fairly approximate the most probable law of the variation of maximum run-off rate with area is not excluded.

B.—Formulas in Which Intensity of Rainfall Does not Appear as a Factor.—The Kuichling formulas:

(*a*) For drainage basins more than 100 sq. miles in area:

$$D_m = M \left(\frac{127\ 000}{M + 370} + 7.4\right) \text{ sec-ft.} \quad \text{..................(40)}$$

and,

$$d'_m = \frac{127\ 000}{M + 370} + 7.4 \text{ sec-ft. per sq. mile} \quad \text{.............(41)}$$

(b) For drainage basins less than 100 sq. miles in area:

$$D_m = M\left(\frac{35\ 000}{M + 32} + 10\right) \text{ sec-ft.} \qquad (42)$$

and,

$$d'_m = \frac{35\ 000}{M + 32} + 10 \text{ sec-ft. per sq. mile} \qquad (43)$$

These formulas are intended to apply to hilly or mountainous regions and to meteorological conditions corresponding to those of the New England and the North Atlantic States. They are not applicable in regions in which rainfall conditions depart materially from those taken into consideration in shaping the formulas.

The Fuller formula:

$$D_m = C\,M^{0.8}\left(1 + 0.8 \log T'\right)\left(1 + \frac{2}{M^{0.3}}\right) \text{ sec-ft.} \qquad (44)$$

and,

$$d'_m = \frac{C}{M^{0.2}}\left(1 + 0.8 \log T'\right)\left(1 + \frac{2}{M^{0.3}}\right) \text{ sec-ft. per sq. mile} \qquad (45)$$

In this formula, T' represents the number of years in which one storm of the intensity which is to be taken into account is probable. This formula, as in the case of all formulas in which the intensity of rainfall does not appear as a factor, has been deduced from run-off data without any attempt to interconnect rainfall intensity and rate of run-off.

The Metcalf and Eddy formula:

$$D_m = 440\,M^{0.73} \text{ sec-ft.} \qquad (46)$$

and,

$$d'_m = \frac{440}{M^{0.27}} \text{ sec-ft. per sq. mile} \qquad (47)$$

This formula was suggested for a Kentucky region and should not be regarded as applicable elsewhere unless topographical and meteorological conditions are similar.

In Fig. 9, there is given in diagrammatic form a comparison of a few formulas to show the conclusions of various investigators as to the effect of area on the maximum rate of run-off. In preparing the diagram, the assumption was made that the rainfall conditions on which each curve is based are the same for all areas, although they were not assumed to be the same for all the curves.

The application of the maximum stream-flow formula can best be made clear by a few examples:

1.—What is the maximum discharge of a river draining a mountain watershed, 1 900 sq. miles in area throughout, on which the maximum rainfall in 24 hours is 8 in. and for which the critical time, $t = 720$ min.? (See Equation (25)).

In this case,

$$a = \frac{60}{60 + 5 \sqrt[3]{720}} = 0.57$$

from Equation (3),

$$C = \frac{8}{24} \sqrt{1\,440} = 12.7$$

from Equation (1),

$$I = \frac{12.7}{\sqrt{720}} = 0.47$$

from Equation (25),

$$D_m = 413 \times 0.57 \times 0.47 \times 1\,900 = 210\ 000 \text{ sec-ft.}$$

The conditions suggested in this example are comparable with those which prevail on the American River, above Folsom, Cal., which, from a water-shed of 1 900 sq. miles, at that point, has probably at its highest known stage discharged as much as 200 000 sec-ft. (1861-62).

2.—What is the maximum discharge of a river draining a water-shed three-fourths of which is mountainous and one-fourth rolling or foot-hill land, 9 000 sq. miles in area, in which the maximum rainfall in 2 days is 6 in. and for which $t = 1\,800$ min.?

In this example, the value of a will lie between the value determined for a mountainous area and that for a rolling country:

$$a = \tfrac{3}{4}\left(\frac{60}{60 + 5 \sqrt[3]{1\,800}}\right) + \tfrac{1}{4}\left(\frac{60}{60 + 20 \sqrt[3]{1\,800}}\right) = 0.42$$

From Equation (3),

$$C = \frac{6}{48} \sqrt{2\,880} = 6.7$$

from Equation (1),

$$I = \frac{6.7}{\sqrt{1\,800}} = 0.158$$

from Equation (25),

$$D_m = 413 \times 0.42 \times 0.158 \times 9\,000 = 247\ 000 \text{ sec-ft.}$$

The conditions suggested in this example are comparable with those which prevail in the water-shed of the Sacramento River above Red Bluff, where the maximum recorded discharge from a drainage basin of 9 300 sq. miles has been about 254 000 sec-ft.

There is no need of extending illustrations to still larger water-sheds, because dependable basic data are not available and because the values of the constants and coefficients here introduced are only tentative. Enough has been said, however, to show that the type of the formula is reasonable. It will be found particularly helpful when from the known conditions of the flow from one water-shed, the maximum rate of run-off or stream flow from another with

similar meteorological and rainfall conditions, is to be estimated. It will also be useful in estimating maximum rates of run-off due to rains of extreme intensity when the maximum rates due to rains of less intensity have been ascertained by stream gauging.

When large portions of a water-shed are lake or reservoir surface, especial consideration may have to be given to the retarding effect of storage in the lakes or reservoirs. The retention of water in storage basins reduces the maximum rate of outflow therefrom and, therefore, reduces the peak of the discharge curve for points below the reservoir. Recourse may be necessary to the mass curve in order to determine this effect.*

Evaporation from Water Surface as Affecting Run-Off.

Whenever any considerable proportion of a water-shed is water surface, as when it embraces lakes of considerable extent, it will become necessary to give the water production due to a known quantity of rain especial study. The depletion of the water bodies or the accession of water which they have received from the beginning of one climatic year to the beginning of the next must be taken into account. This may involve a study of the loss of water by evaporation. When the rainfall is light, this loss of water from a lake by evaporation frequently exceeds the accession resulting from rain falling directly on the water. The water production of the lake area under such circumstances is negative. In computing the evaporation from the surface of a lake or reservoir, the following simple relation between mean monthly temperature and the probable average rate at which water is lost by evaporation throughout the month will be found to be helpful. This relation between the mean monthly temperature of the atmosphere and the rate of evaporation is shown in Table 9.

TABLE 9.—Evaporation from Open Water Surface.†
(Based on a Revised Curve.)

Mean monthly temperature, in degrees Fahrenheit.	Rate of evaporation, in feet per day.	Evaporation per Month:		
		In 28 days. Inches.	In 30 days. Inches.	In 31 days. Inches.
25	0.0015	0.50	0.54	0.56
30	0.0019	0.64	0.68	0.71
35	0.0025	0.84	0.90	0.93
40	0.0033	1.12	1.19	1.23
45	0.0043	1.48	1.55	1.60
50	0.0056	1.88	2.02	2.06
55	0.0073	2.45	2.63	2.71
60	0.0095	3.19	3.42	3.53
65	0.0121	4.06	4.32	4.50
70	0.0151	5.07	5.44	5.62
75	0.0186	6.25	6.70	6.92
80	0.0223	7.50	8.04	8.30
85	0.0260	8.74	9.37	9.68
90	0.0300	10.08	10.80	11.16

It seems to be fairly well established that evaporation increases somewhat with elevation above sea level. In the absence of any conclusive data relating

* *Transactions*, Am. Soc. C. E., Vol. LXI (1908), p. 335.

† *Transactions*, Am. Soc. C. E., Vol. LXXX (1916), p. 1968.

to the rate of this increase, the following formula is suggested for use until something better is offered:

$$E = E' + 0.0012 \sqrt{F H} \dots\dots\dots\dots\dots\dots\dots (47)$$

In this formula, E is the monthly evaporation, in inches, at the place for which evaporation is to be determined, when the altitude of the place is H ft. above sea level; and E' is the monthly evaporation at sea level given in Table 9, which would obtain at the mean monthly temperature of F degrees, noted for any month at the place at which evaporation is to be estimated.

It must not be expected that for any single month evaporation will always be indicated correctly by the evaporation curve. Owing to the great variation in conditions of wind, humidity, sunshine, and the daily range of temperature, there may be wide departures from the monthly evaporation rate which the curve indicates as the probable rate. For 12 months, the probable error is much less than for any single month. The curve is intended for use when the annual evaporation is to be determined.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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DISCUSSION ON TENTATIVE SPECIFICATIONS FOR CONCRETE AND REINFORCED CONCRETE

SUBMITTED AS A PROGRESS REPORT OF THE JOINT COMMITTEE ON STANDARD SPECIFICATIONS FOR CONCRETE AND REINFORCED CONCRETE

BY W. A. SLATER, ASSOC. M. AM. SOC. C. E.

W. A. SLATER,* Assoc. M. Am. Soc. C. E. (by letter).†—As Chairman of the representatives of the Society on the Joint Committee on Standard Specifications for Concrete and Reinforced Concrete, the writer wishes to state that the Progress Report of the Joint Committee was published‡ with the provision in mind that there are certain parts of it with which the representatives of the Society individually do not agree. It appeared to be the general opinion, however, that better progress would be made by submitting the report for discussion rather than delay publication in an endeavor to bring all the members of the Joint Committee to complete agreement. It is expected that unpublished data which afford the basis for certain provisions of the specifications will be published as separate papers by individuals in the *Proceedings* and *Transactions* of the technical societies, or other technical papers, as rapidly as possible.

It is the writer's aim in the following statement to indicate the basis for some of the essential features of the proposed flat-slab specifications§ included in Sections 145 to 162, of the Progress Report of the Joint Committee (herein termed the Progress Report), and the manner in which available data justify these specifications. The statement is limited to discussion of the basis for the proposed (1) moment coefficients for design: (2) formulas for slab thickness; and (3) formulas for compressive stress.

* Washington, D. C.

† Received by the Acting Secretary July 27th, 1921.

‡ *Proceedings*, Am. Soc. C. E., August, 1921, p. 59.

§ *Proceedings*, Am. Soc. C. E., August, 1921, p. 97.

A comparison of the Final Report* of July 1st, 1916, by the Joint Committee on Concrete and Reinforced Concrete (herein termed the J. C. 1916 report), with the results of analysis on one hand and the results of tests on the other, indicates that in most essentials that report represents substantially the conclusions which must be reached after a study of whatever information has been accumulated on the subject of flat slabs.

The J. C. 1916 report, therefore, has been made the basis of the present specifications, but with some departures which are important enough to have the reasons therefor indicated.

Moment Coefficients.—The former Joint Committee recognized that the sum of the positive and negative moments in the direction of each side of the panel was approximately:

$$M_0 = \frac{1}{8} W l \left(1 - \frac{2}{3}\frac{c}{l}\right)^2 \qquad (51)\dagger$$

but believed that the results of tests warranted the use of a total moment for design approximately 15% less than that value. Since the J. C. 1916 report was made, an analysis by Mr. H. M. Westergaard‡ confirms the correctness of Equation (51) and indicates in considerable detail how the moments are distributed in various parts of the slab.

In the same investigation a study of the available data of tests of flat slabs has been made, in which there was developed an approximate method of evaluating the amount of tensile stress carried by the concrete. When the moment of the tensile stresses in the concrete is added to the moment of the tensile stress observed in the reinforcement, the total moment is found to approach the value determined analytically as the sum of the positive and negative moments, as given in Equation (51).

However, the study indicates that, as a mean value, the factor of safety for the tests referred to is sufficient to warrant reducing the moment for which the reinforcement must be designed to about 28% less than the moment determined analytically, that is, to Formula 37§ of the Progress Report, as follows:

$$M_0 = 0.09\, W l \left(1 - \frac{2}{3}\frac{c}{l}\right)^2$$

On account of the fact that very few tests of flat slabs have been carried far enough to cause failure, it was necessary, for the purposes of the investigation referred to, to use estimated values of the factor of safety based on an analogy between the hehavior of beams and the behavior of slabs. That the estimated values of the factor of safety are conservative is evidenced by the fact that the highest values given are for the structures in which the test was carried to destruction, or nearly so. Further confidence in the conclusion arrived at in this manner is added by the fact that, in a recent test of a slab supported on girders on the four edges, the reserve strength which was

* *Transactions*, Am. Soc. C. E., Vol. LXXXI (1917), p. 1101.

† For notation see *Proceedings*, Am. Soc. C. E., August, 1921, p. 97.

‡ "Moments and Stresses in Slabs", by H. M. Westergaard and W. A. Slater, Assoc. M. Am. Soc. C. E., *Proceedings*, Am. Concrete Inst., Vol. XVII (1921).

§ *Proceedings*, Am. Soc. C. E., August, 1921, p. 97.

developed after the yield point of the steel had been passed, was much greater than that of reinforced concrete beams tested in the laboratories.

In judging of the propriety of such a moment coefficient as that proposed, it seems proper to consider the fact that the Building Department of Chicago, Ill., specifies a coefficient which is 2% greater for two-way and 4% less for four-way slabs than the one here proposed. The municipal authorities of Chicago have generally taken a conservative attitude on the flat-slab question and have proceeded with flat-slab regulations only on the basis of tests made for the purpose of furnishing data which would serve as a guide in the preparation of flat-slab regulations. The fact that the ruling has been renewed at various times without increase of the coefficient is an indication that it has not been found to be too liberal in its provisions.

Slab Thickness.—An important feature of the formula for slab thickness is the fact that it gives thicker slabs than would be obtained by using the moment coefficient given Section 146* of the Progress Report in conjunction with the specified working stress in compression and with the usual method of computing compression. This extra thickness is provided in order to keep down the compressive stress, although it is recognized that in most flexural members the factor of safety against compression failure in the concrete is much greater than the factor of safety in tension. This may be as true of flat slabs as of simple beams, since the percentage of reinforcement is usually low in flat slabs; but there are other factors which indicate the desirability of keeping the compressive stresses as low as provided by Formula 38† of the Progress Report, namely:

(1) The tensile stresses in the steel would be relieved because of the tensile stresses resisted by the concrete, but the compressive stresses in the concrete would be very little affected thereby. Allowance of some kind should be made for this fact.

(2) It has been brought out in many cases that there is a progressive yielding of the concrete‡ under constant load, which results in a progressive deflection, and in fixing specifications it seems desirable to consider the necessity of avoiding excessive deflection as well as of securing a sufficient factor of safety against actual failure. Although tests have given indications as to strength, they have not shown the conditions under which excessive deflection is likely to occur.

(3) Although for convenience in design, computations of compressive stress are made as though the stress due to the critical moment per unit width in the column strip is the same at all points in the width of the strip, it is recognized that this assumption is incorrect. "Observations and tests indicate that the maximum stress in the column head section (here the column strip) may be

* *Proceedings,* Am. Soc. C. E., August, 1921, p. 97.

† *Proceedings,* Am. Soc. C. E., August, 1921, p. 98.

‡ A. R. Lord, "Extensometer Measurements in a Reinforced Concrete Building Over a Period of One Year", *Proceedings*, Am. Concrete Inst., Vol. XIII (1917), p. 45; F. R. McMillan, M. Am. Soc. C. E., "Shrinkage and Time Effects in Reinforced Concrete", Studies in Engineering, Univ. of Minnesota, No. 3 (1915); *Journal*, Engrs.' Club of St. Louis, Vol. I, No. 3; *Engineering News*, March 11th, 1915; E. B. Smith, "Flow of Concrete under Sustained Loads", *Proceedings*, Am. Concrete Inst., Vol. XII, p. 317 (1916), and Vol. XIII, p. 99 (1917); A. H. Fuller, M. Am. Soc. C. E., and C. C. More, Assoc. M. Am. Soc. C. E., "Time Tests of Concrete", *Proceedings*, Am. Concrete Inst., Vol. XII, p. 302 (1916).

expected to be 25% more than the average stress in the column head section, or even higher".* The analysis by Mr. Westergaard† indicates that in a homogeneous slab without dropped panel, the maximum negative moment per unit width at the end of the column strip is approximately 80% greater than the average moment per unit width on the same section when $\frac{c}{l} = 0.225$, and that the relation of maximum to average is approximately:

$$r = 2.875 \left(1 - 1\frac{2}{3}\,\frac{c}{l}\right) \qquad (52)$$

where r = ratio of maximum to average moment per unit width.

For the reasons here stated, Formula 38, Section 148,‡ of the Progress Report, has been proposed, which gives a greater slab thickness than would result from the use of the moment requirements of Section 146 of the same report and the ordinary methods of computing the compressive stress.

The J. C. 1916 report provides for a greater difference between the negative moment in the column strip for slabs with dropped panels and that for slabs without dropped panels than the analysis by Mr. Westergaard§ seems to indicate as necessary. Accordingly, the proposed specifications require thicknesses which are more nearly the same for slabs with and slabs without dropped panels than those provided for by the J. C. 1916 report. This is brought out in Fig. 19, in which is given the coefficient of $l\sqrt{w'}$ in the slab thickness formulas required by (1) the former Joint Committee, (2) the present Progress Report, and (3) the Chicago Building Department. The latter is taken from a ruling dated March 1st, 1918.

The values of R used for Formula 38, of the Progress Report, in Fig. 19, are those recommended in Table 3.‖ The tolerances permitted are not shown. The values of $\frac{l_1}{b_1}$ are taken as 2.5 and 2.0, respectively, for the slabs with and for those without dropped panels. This is according to the requirements of the J. C. 1916 report.

It will be seen that for slabs with four-way reinforcement and with dropped panels, the thickness required by Formula 38 of the Progress Report, is the same as that required by the J. C. 1916 report, namely:

$$t = 0.03\,l\sqrt{w'} + 1\tfrac{1}{2} \qquad (53)$$

when $\frac{c}{l}$ is 0.225, which is a common value in practice. For larger capitals, thinner slabs are permitted by the Progress Report, while for smaller capitals considerably thicker slabs are required. It is believed that this greater thickness is sufficient to prevent the use of extremely small capitals in most cases. There are, however, some cases in which small capitals are required regard-

* A. N. Talbot, Past-President, Am. Soc. C. E., *Proceedings,* Am. Concrete Inst., Vol. XIII, p. 406 (1917).

† "Moments and Stresses in Slabs", *Proceedings,* Am. Concrete Inst., Vol. XVII, p. 443 (1921).

‡ *Proceedings,* Am. Soc. C. E., August, 1921, p. 98.

§ *Proceedings,* Am. Concrete Inst., Vol. XVII (1921).

‖ *Proceedings,* Am. Soc. C. E., August, 1921, p. 98.

less of the cost, and for these cases the introduction of the term, $\frac{c}{l}$, into Formula 38 of the Progress Report provides a safeguard against excessive concentration of compressive stresses within the width of the column capital.

It is realized that the thickness (Formula 38) appears to be formidable, but it should be recognized that this is more than a lower limit formula. When the requirements of Formula 38 are met, one can feel reasonably certain that the requirements for compression are met economically as far as compliance with the specifications are concerned. With the requirements of the Chicago Building Department and the J. C. 1916 report, it is still necessary, after the thickness formulas are complied with, to design the slab for tension and com-

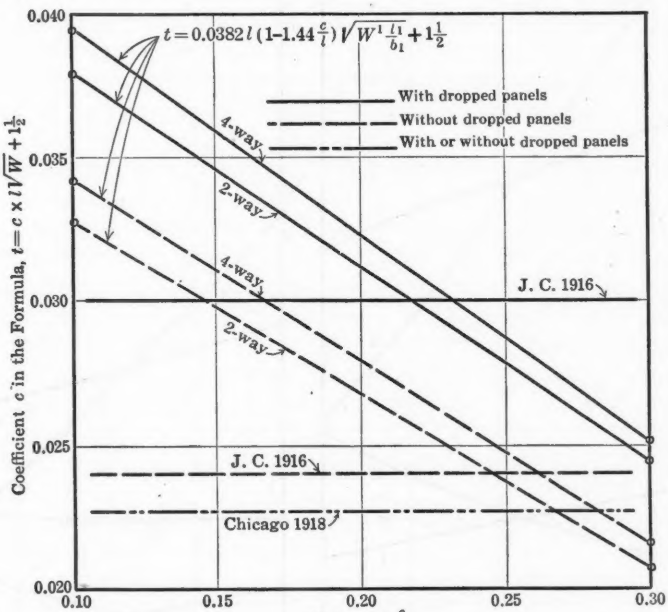

FIG. 19.

pression requirements. With this fact in mind, it does not seem that Formula 38 adds to the burden in designing.

Compressive Stresses.—The computed compressive stresses due to the critical negative moment in the column strip are shown in Fig. 20. All the stresses shown are for slabs with dropped panels. For the slab thicknesses given by Formula 38 of the Progress Report, the compressive stresses will be the same, however, whether or not a dropped panel is present. This is so nearly true for the thicknesses and moment distributions recommended by the J. C. 1916 report, that the stresses computed for slabs with dropped panels suffice also for the consideration of slabs without dropped panels. The value of $\frac{l_1}{b_1}$ for Fig. 20 is 2.5 in all cases.

Although in Section 146 of the Progress Report, a considerable range in assumptions is permitted as to the proportion of the total moment, M_0, which is

to be resisted as negative moment in the column strip, the computed compressive stress is not affected, since the term, R, which represents that proportion is involved in the slab thickness, Formula 38, as well as in the negative moment in the column strip and cancels in computing the compressive stress. In order to give a closer approximation to the true stresses than those obtained by the use of the arbitrary coefficient, 0.09, in Equation (52), and the corresponding coefficient, 0.1067, from the J. C. 1916 report, Fig. 20 (a) shows the stresses

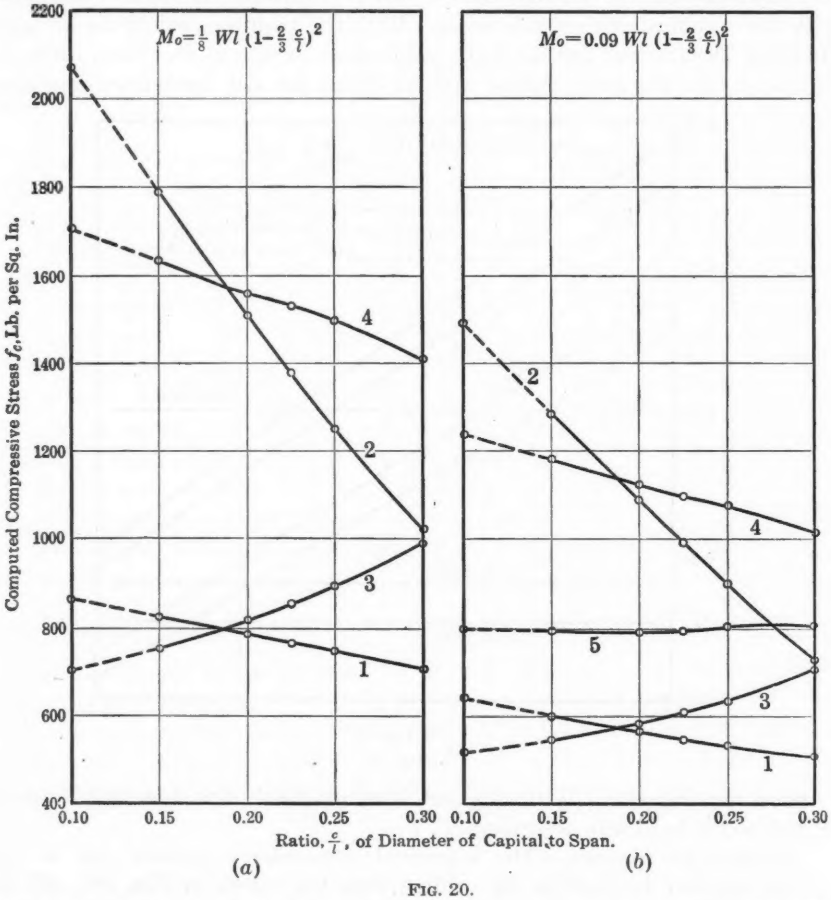

Fig. 20.

changed to the basis of the full moment, M_0, as given in Equation (51). The stresses given in Curve (1), Fig. 20 (a), are $\frac{0.125}{0.1067}$ times as great as would be obtained by using directly the recommendations of the J. C. 1916 report for slab thickness and compressive stress. The values in Curve (2), Fig. 20 (a), are $\frac{0.125}{0.09}$ times as great as would be obtained by using Formula 37 of the Progress Report, together with the equation:

$$f_c = \frac{2\, R\, M_0}{k\, j\, b_1\, d^2} \quad \text{.......................} (54)$$

for average compressive stress within the width of the dropped panel, in which b_1 is the width of the dropped panel.

For convenience of computation, with values of $p\,n$ between 0.04 and 0.24, $k\,j$ may, with very slight error, be stated as:

$$k\,j = 0.67\,\sqrt[3]{p\,n} \quad \ldots\ldots\ldots\ldots (55)$$

The value of p necessary to keep the tensile stress at 16 000 lb. per sq. in., when Formulas 37 and 38 of the Progress Report are used, is given closely by the equation:

$$p = 0.0045 + 0.048\left(\frac{c}{l}\right)^2 \quad \ldots\ldots\ldots\ldots (56)$$

Equations (55) and (56) are of some importance since the values of p, k, and j cannot be determined from the usual relation between the tensile and the compressive stress. This is because the stresses are not the average stresses generally used. Values of k and j are given correctly by the ordinary formulas involving p and n.

The stresses given in Curves (2) and (4) of Fig. 20 (*a*) and (*b*) are intended to represent the maximum stresses, that is, those within the width of the column capital. They were obtained by multiplying the stresses shown in Curves (1) and (3) by the values of r obtained from Equation (52). That equation is not rigidly applicable since it was derived for slabs without column capitals and gives the ratio of the maximum moment per unit width to the average within the column strip. The use of it in these computations is equivalent to assuming that the distribution within the width of the dropped panel is similar to that within the width of the column strip when there is no dropped panel.

Curves (1) to (4) in Fig. 20 (*b*) are similar to those of Fig. 20 (*a*), except that in the latter M_0 is taken the same as in Formula 37 of the Progress Report, instead of Equation (51). Curve (5) gives the nominal stress computed by Formula 41* of the Progress Report:

$$f_c = \frac{3.5\,R\,M_0}{k\,j\,b_1\,d^2}\left(1 - 1.2\,\frac{c}{l}\right)$$

If the values of M_0 and d from Formulas 37 and 38 are substituted in Formula 41, the latter reduces to:

$$f_c = \frac{216}{k\,j}\left(\frac{1 - \frac{2}{3}\,\frac{c}{l}}{1 - 1.44\,\frac{c}{l}}\right)^2\left(1 - 1.2\,\frac{c}{l}\right) \quad \ldots\ldots\ldots\ldots (57)$$

It is recognized that the stresses given by Curve (5) of Fig. 20 (*b*) are not the true stresses, but it is believed that they are more nearly equal to the true stresses than the average stresses usually computed in flat-slab design.

The constants in Formula 41 have been determined so that when that formula is used in computing compressive stresses in a slab of a thickness which has been generally found to be adequate, the same compressive stress is found for all values of $\frac{c}{l}$ and the same for flat slabs as the specifications allow

* *Proceedings*, Am. Soc. C. E., August, 1921, p. 101.

for other forms of structures. The same result could have been obtained by specifying different working stresses for flat slabs than for other structures and a different allowable compressive stress for each different size of column capital, but the method proposed was believed to be preferable. The original intention was that the stresses by this formula should be the same proportion of the maximum compressive stress, for all diameters of column capital. This intention is not fully realized since, with the slab thickness used, the maximum compressive stresses are not the same (for the reason indicated subsequently) for all sizes of column capital.

The aim in deriving a formula for thickness of slabs was to fix the constants in such a way that for all values of $\frac{c}{l}$ within the ordinary range of practice the maximum stress due to negative moment in the column strip (rather than the average) would be the same. The formula proposed meets this condition closely when the compressive stresses are computed, assuming $k\,j$ at a constant value, but when the $k\,j$ corresponding to the correct percentage of reinforcement is used, a larger variation in the maximum stress resulted than was anticipated. However, by comparing Curves (2) and (4), Fig. 20 (*a*), it will be seen that a marked evening up of the stresses has been effected by introducing the term $\left(1 - 1.44\,\frac{c}{l}\right)$ into the slab-thickness formula. Since the analysis underlying the distribution of moments involved in Formula 38 of the Progress Report was made for homogeneous slabs without column capitals, and since in order to apply it to slabs having column capitals, it was assumed that the proportionate moment distribution within the width of the dropped panel was the same as that within the entire column strip for slabs without dropped panels, it did not seem that there was sufficient certainty of increased accuracy to warrant a further modification of the slab-thickness formula. In fact, it seems quite possible that the greater yielding of the concrete where the stress is greatest would be sufficient to flatten out the curve of maximum compressive stress still more than it has been.

Requiring the stress by Formula 42* of the Progress Report to be not greater than 800 lb. per sq. in., is equivalent to requiring an average stress not greater than 740 lb. per sq. in. when computed with M_0 according to Equation (51). This may be shown in Equation (58), as follows:

$$f_c = \frac{2\,M}{k\,j\,b\,d^2} = \frac{2\,R\,M_0}{k\,j\,\frac{l_1}{2}\,d^2} \times \frac{0.125}{0.09} \times \frac{800}{740} = \frac{6\,R\,M_0}{k\,j\,l_1\,d^2} \qquad (58)$$

since, under these circumstances, M will be equal to $\frac{0.125}{0.09} \times \frac{800}{740}\,R\,M_0$, and b will be equal to $\frac{l_1}{2}$.

This requirement is rather more severe than that for compressive stresses due to negative moment, and although the compression, even by this formula is not likely in many instances to reach 800 lb. per sq. in., it is possible that it should be modified.

* *Proceedings*, Am. Soc. C. E., August 1921, p. 101.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

EDMUND TAYLOR PERKINS, M. Am. Soc. C. E.*

DIED MAY 21ST, 1921.

Edmund Taylor Perkins was born in Scottsville, Va., on September 8th, 1864. His father was Edmund Taylor Perkins, of Virginia, and his mother, Mary Addison, of Maryland. He was graduated from Union College, Schenectady, N. Y., in 1885, with degrees of C. E. and A. B., and received the degree of M. A., in 1888.

Mr. Perkins served with the U. S. Geological Survey from 1885 to 1902. Later, he was with the U. S. Reclamation Service (1902-1909) in charge of stream gauging and the determination of run-off factors in Colorado, and of the preliminary surveys and plans for the Yuma Irrigation Project in Arizona; he also served as General Inspector of the Chicago Transportation and Contracting Office.

In 1910, he established the present office of the Edmund T. Perkins Engineering Company, in Chicago, Ill. As a Consulting Engineer, Mr. Perkins specialized in land drainage, reclamation, and flood control. He was a colleague of the late Isham Randolph, M. Am. Soc. C. E., and with him served as a member of the Everglades Engineering Commission of the State of Florida. He was Engineer for the Green Bay Levee and Drainage District No. 2 of Lee County, Iowa; the South Quincy and Valley City Drainage Districts of Illinois; the Marion County Drainage District of Missouri; and also acted as Consulting Engineer for several large drainage districts of Arkansas.

He had just returned from a trip to South America where he had been engaged on an investigation and report of the resources of Colombia with a view to extensive development there, when he suffered a sudden attack of heart failure from which he died at his home in Chicago, Ill., on May 21st, 1921. He was buried at his boyhood home in Louisville, Ky. Mr. Perkins is survived by his wife, Louise Scribner Perkins, formerly of Washington, D. C., and two sisters.

He was an active and loyal worker in the engineering societies of which he was a member. He was President of the National Drainage Congress which he organized on December 8th, 1920; and a member of the Board of Directors and Past-President of the American Association of Engineers. He was also a member of the Western Society of Engineers, the Illinois Society of Engineers, and the Engineers' Clubs of New York City and Chicago.

Mr. Perkins was also active in the work of the Mississippi Valley Association and the Chicago Association of Commerce, and was a member of the University Club, Iroquois Club, and Glen View Club of Chicago, and the Chevy Chase Club of Maryland.

* Memoir prepared by L. K. Sherman, M. Am. Soc. C. E.

He was a man of genial disposition and won many friends. He believed that the engineer should take his part in the duties of citizenship and public affairs.

Mr. Perkins was elected a Member of the American Society of Civil Engineers on December 3d, 1902.

GEORGE STAPLES RICE, M. Am. Soc. C. E.*

Died December 7th, 1920.

George Staples Rice was born in Boston, Mass., on February 28th, 1849. He came of sterling New England stock, his ancestors having borne their part in constructing the history of New England in the old Colonial days. His father was Reuben Rice, a direct descendant of Edmund Rice, who emigrated from England to the United States in 1638, and was one of the early settlers of Marlborough, Mass. His mother, Harriet Tyler (Ketelle) Rice, also came of old English ancestry.

His early education was received in the public schools of Boston. Afterward, he entered Harvard University and was graduated in 1870, with the degree of S. B.

Even before his course at Harvard was finished, Mr. Rice began his public work. At that time, municipal engineering in the United States was just beginning. Previous to his last year at the University, Mr. Rice spent the summer in the service of the Engineering Department of the Boston Water-Works, assisting in the construction of the Chestnut Hill Reservoir. After his graduation from Harvard, he became Assistant Engineer of the Lowell, Mass., Water-Works, and, later, Assistant Division Engineer of the Boston Water-Works. From 1877 to 1880, he filled the position of Assistant Engineer and Principal Assistant Engineer in charge of the Boston Main Drainage Works, which was one of the most important sanitary engineering projects ever undertaken by the City of Boston.

At this time, the late James B. Francis and Alphonse Fteley, Past-Presidents, Am. Soc. C. E., and Joseph P. Davis, M. Am. Soc. C. E., all pioneer hydraulic engineers, were practising their profession in Boston, Mr. Davis having been City Engineer of Boston during this time. Hiram F. Mills, Hon. M. Am. Soc. C. E., also belonged to this group. Daily contact and association with such men was an inspiration to Mr. Rice, then a young engineer, and greatly influenced his whole professional career.

In 1880, however, the lure of the West became too strong for him, and he went to Arizona and Colorado where he was engaged in mining operations. This work occupied him for seven years, when the call came for him to return East.

His previous experience on the public works of Boston brought him the appointment, in 1887, of Deputy Chief Engineer of the Aqueduct Commission of New York City. Thus, he began his work for the city to which he was destined to give the greater part of his life's service.

* Memoir prepared by D. L. Turner, M. Am. Soc. C. E.

In 1891, Mr. Rice returned to his native city to become Chief Engineer of the Boston Transit Commission. He made a thorough investigation into the transit conditions of the city, and prepared a comprehensive report which was an important factor in the future development of the transit facilities of Boston. He remained in Boston until 1900. During this period (1891-1900), he was also engaged in the private practice of engineering with the late George E. Evans, M. Am. Soc. C. E., under the firm name of Rice and Evans. This firm designed and constructed the water-works of New Bedford, Mass.

While practising his profession in Boston, Mr. Rice received a call to aid his Alma Mater in developing its School of Engineering from which he had been graduated twenty years before, and to this call he gladly and loyally responded. For eight years (1892-1900), Mr. Rice served Harvard University as Instructor in Sanitary Engineering.

Meanwhile, his work for the Boston Transit Commission had attracted attention in New York City where improved transit facilities had become a vital necessity. William Barclay Parsons, M. Am. Soc. C. E., Chief Engineer of the Board of Rapid Transit Railroad Commissioners, appointed him Deputy Chief Engineer of the Commission, and he served with Mr. Parsons during the construction of the first subway for New York City. When Mr. Parsons resigned on December 31st, 1904, Mr. Rice succeeded him, first as Acting Chief, and then as Chief Engineer, of the Commission.

In 1910, shortly after the Public Service Commission succeeded to the work of the Rapid Transit Board, Mr. Rice became Engineer of Subway Construction, but he resigned this position the same year and went into private practice. Later, in 1914, when the work of constructing the Dual Subway System for New York was begun, he returned to the service of the City and became a Division Engineer in charge of the construction of a large part of the work, which position he held until his death on December 7th, 1920.

Mr. Rice served his native city, Boston, and his adopted city, New York, in a highly honorable and unselfish manner throughout thirty years of his professional career; in other words, he devoted nearly two-thirds of his professional life to the public service.

He was married in Yonkers, N. Y., on October 10th, 1889, to Rose Breuchaud, who with one son, Albert F. Rice, survives him.

His long and active life brought him many friends, to all of whom he was greatly endeared because of his lovable and unselfish character.

At his death, the Faculty of Harvard University adopted the following minute:

"George Staples Rice, S. B. 1870, was a most loyal and useful friend of Harvard University. He rendered distinguished service to the University by a highly honorable and unselfish career as the director of great public works vitally affecting the welfare and safety of millions of people, included in which are some of the most important engineering enterprises of his time, such as the present water supply, drainage, and rapid transit systems of Boston, the New Croton Aqueduct, and the great subway system of New York.

"Mr. Rice served the University devotedly as Instructor in Sanitary Engineering for a period of eight years (1892-1900), after the close of his work as

Deputy Chief Engineer for the New Croton Aqueduct—in the full tide of his life of great achievement.

"Mr. Rice's official relations with the University terminated when he was called to New York to become Deputy Chief Engineer, and soon after, Chief Engineer of the New York subway development; but this did not close his direct co-operation with Harvard work in Engineering. Through his foresight, his friendly advice, and encouragement, in the early days of the New York Subway, he made his own opportunities count as opportunities also for scores of younger men from Harvard. Thus, he, perhaps above all others of our graduates for the past fifty years, opened the way by which the University, through these young men, is steadily extending its influence for the convenience and safety of the public, and by which the young men themselves are securing the lasting satisfaction which comes from doing great and worthy tasks well.

"The Faculty of Engineering accordingly and with deep gratitude records its appreciation of the inestimable value of Mr. Rice's energetic and unceasing loyalty to Harvard University."

He was a member of the Boston Society of Civil Engineers, the American Institute of Mining and Metallurgical Engineers, the New England Water-Works Association, and the Society of Colonial Wars. His clubs were the University, Harvard, and Arkwright of New York City, and the Union and St. Botolph of Boston, Mass.

Mr. Rice was elected a Member of the American Society of Civil Engineers on February 1st, 1882.

GEORGE SYKES, M. Am. Soc. C. E.*

Died August 21st, 1919.

George Sykes was born on April 5th, 1881, in New York City, where he received his early education.

He began his engineering experience in 1901, as a Leveler in the employ of the Guayaquil and Quito Railway for a period of about nine months, before completing his technical education. He then returned to school and, in June, 1903, was graduated from the School of Applied Science of New York University with the degree of B. S. in Mechanical Engineering.

Before his graduation Mr. Sykes entered the employ of the United Electric Light and Power Company, of New York City, as an Inspector on electric transformer building work, and immediately after his graduation continued in the same capacity with the New York Edison Company where his construction experience included the building of office buildings and storehouses in New York City and the erection of a large coal storage plant with a river bulkhead at Edgewater, N. J.

In July, 1907, Mr. Sykes organized a corporation for general construction work and for ten years conducted a very successful contracting business. His field was confined largely to New York City and vicinity, but his most important operation was the building of "Vizcaya", the residence of Mr. James Deering, at Miami, Fla.

* Memoir prepared by E. M. Van Norden, M. Am. Soc. C. E.

When the United States entered the World War, he was commissioned as Captain in the Engineer Officers' Reserve Corps, on July 12th, 1917, and attended the training camp as a student at American University, Washington, D. C. He was ordered to France, in September, 1917, and served as Executive Officer for the Chief Engineer of Line of Communication, as Chief of the Administration Section, in the office of the Director of Construction and Forestry, Service of Supplies, and as Engineer Officer in Chief, Divisional Area Construction, Zone of Advance, A. E. F. He was promoted to the rank of Major, Engineers, U. S. A., on July 12th, 1918, and after his return to the United States, he was honorably discharged at Washington, D. C., on January 21st, 1919.

Maj. Sykes' patriotism resulted in a tremendous sacrifice to himself and his family which, while it may be true of many other engineers, is none the less deserving of mention; he gave up a prosperous contracting business and spent sixteen months in France away from his wife and children. His military ambitions were probably inherited from his grandfather who was Maj.-Gen. George Sykes, Commander of the 5th Army Corps, Army of the Potomac.

After his discharge from the Army, Maj. Sykes resumed his position as President of the George Sykes Building Company in New York City, and was actively engaged in business until a week before his death on August 21st, 1919, after an operation at the Flushing Hospital. He is survived by his mother, his wife, and three children.

He was an active member of the Bayside, N. Y., Yacht Club and the Delta Phi Fraternity.

Maj. Sykes was elected a Junior of the American Society of Civil Engineers on May 1st, 1906, an Associate Member on December 4th, 1907, and a Member on May 12th, 1919.

JOSEPH MILLER BURKETT, Assoc. M. Am. Soc. C. E.*

Died April 26th, 1921.

Joseph Miller Burkett, the son of Joseph Miller Burkett and Ida Adelle Pinney Burkett, was born in Salt Lake City, Utah, on March 30th, 1880. His father was long associated with the pioneer banking firm of McCornick and Company, in Salt Lake City, but, in 1882, moved with his family to Hailey, Idaho, where he opened the First National Bank of that city. Mr. Burkett's early childhood was spent in Hailey which, at that time, was an active and prosperous mining camp, so it was only natural that his first engineering efforts were in connection with the mining industry.

In 1890, he entered the Ogden Military Academy, at Ogden, Utah. After attending the Academy for three years he went to Boise, Idaho, and entered the public schools of that city. While in Boise, Mr. Burkett resided with his uncle, Mr. James A. Pinney, one of the well known business men and early pioneers of Idaho. On the completion of his public school education, he entered Oberlin College, in Oberlin, Ohio, where he remained for three years.

* Memoir prepared by W. G. Swendsen, Assoc. M. Am. Soc. C. E.

While at Oberlin, he spent his summer vacations selling books and following similar pursuits common to the more ambitious students of that day. Desiring to engage in the Engineering Profession, Mr. Burkett then entered the Leland Stanford, Jr., University, in California, and while there was engaged during his summer vacations in various capacities with field crews on surveys for the Santa Cruz Railroad.

After leaving Stanford, Mr. Burkett returned to Idaho and began the practice of engineering. At first, he was engaged principally in mining work, but owing to the rapid irrigation development of Southern Idaho about this time, he became occupied with reclamation work almost exclusively and spent several years in active field work of various kinds. Although a considerable part of his practice was located in Idaho, he also did more or less work in Montana and Nevada and, at one time, was interested in a large irrigation project in Texas.

In 1911, Mr. Burkett entered the employ of the State of Idaho as Carey Act Engineer, serving under A. E. Robinson, State Engineer, and continuing in the employment of the late Frank King who succeeded Mr. Robinson. In the capacity of Carey Act Engineer, he made various investigations and generally supervised the construction work on practically all the principal Carey Act projects in Idaho. In 1915, he resigned his position with the State and entered into private practice, locating at Twin Falls, Idaho. While maintaining his office in that city, he conducted a general engineering business, having had charge of considerable municipal work for the various communities that were rapidly building up in Southern Idaho.

When the United States entered the World War, Mr. Burkett at once volunteered and was appointed a Captain of Engineers. After a short period of training at Vancouver, Wash., he was immediately sent overseas with the 116th Engineers, with which he served as Captain throughout the war. He was located at various places in France for nearly two years, and was honorably discharged after the Armistice was signed.

On his return from France, Mr. Burkett again took up the practice of his Profession, with Boise, Idaho, as his headquarters. His practice while in Boise was confined almost altogether to the supervision of irrigation developments throughout the State, having served as Chief Engineer on some five or six projects in various stages of completion. He also acted as Consulting Engineer to the Commissioner of Reclamation of Idaho in connection with various irrigation problems coming before the Department of Reclamation, and, acting under the Commissioner, he conducted engineering investigations incident to several important river adjudication suits, the principal among which were those relating to the Portneuf and Pahsimeroi Rivers.

Owing to his extensive knowledge of irrigation developments in Idaho, and his broad experience in irrigation matters generally, Mr. Burkett was often called on for expert testimony in connection with litigations which occurred from time to time, and in this capacity was invariably regarded as an engineer of the highest ability and integrity.

He died on April 26th, 1921, at Pocatello, Idaho, after a hurried operation for appendicitis, to which he submitted while on a field trip, and was buried

at Boise. He was not married, and is survived by his mother who lives in Hailey, and an only sister, Mrs. T. M. Starrh, of Boise. He was a member of the Masonic order and of the Benevolent and Protective Order of Elks.

Mr. Burkett was elected an Associate Member of the American Society of Engineers on May 28th, 1912.

JOHN EDWARD GRADY, Assoc. M. Am. Soc. C. E.*

Died May 19th, 1921.

John Edward Grady was born in Kenosha, Wis., on May 10th, 1874. He received his common school education in Chicago, Ill., finishing two years of High School and a short course in a business college in December, 1890.

At the age of sixteen, he went to work as a Timekeeper for the Kelly Construction Company, General Contractors, thus beginning his engineering education in the school of experience. In April, 1893, he was engaged as a Rodman on the location of the Chicago Sanitary and Ship Canal and continued in the employ of The Sanitary District of Chicago until June, 1906, when he left of his own accord to join the Great Lakes Dredge and Dock Company.

During the thirteen years (1893-1906) of his employment with the Sanitary District of Chicago, Mr. Grady progressed steadily, and grew in character, ability, and attainments. In 1894, he was variously employed on canal construction. In 1895 and 1896, he was engaged in the Computing Department on preliminary and final estimates, progress charts, and construction reports. In 1897, he was put in charge of the Computing Department and, in 1899, he was given charge of the construction of the Chicago River By-Pass under the Pennsylvania Railroad Company's properties at the Chicago Union Station. In 1901, he was put in charge of bridge construction across the Chicago River and, during the ensuing five-year period, he completed the building of five bascule bridges at an aggregate cost of $1 125 000.

It was during this period that the writer had the privilege of serving under Mr. Grady and of learning, by daily intercourse, his fine qualities of mind and heart. He possessed, in a marked degree, the qualities of leadership as evidenced by the devotion of his subordinates, the friendly admiration of his equals, and the respect and confidence of his superiors.

During the thirteen years (1906-1919) of his employment with the Great Lakes Dredge and Dock Company, Mr. Grady occupied a position of increasing responsibility. His first work for this Company consisted of the engineering supervision of the extensive dock, harbor, and foundation work for the United States Steel Corporation, at Gary, Ind. While still conducting this work, he was made Division Engineer for the Company, with headquarters at Cleveland, Ohio, and had charge of its extensive interests in the Cleveland Division. He installed the large plant at Cleveland for the manufacture of premoulded concrete piles which were made in large quantities and first used in the extensive ore docks built for the Pennsylvania Railroad Company in that

* Memoir prepared by Robert Isham Randolph, M. Am. Soc. C. E.

city. He also planned and built the foundations for the High Level Bridge at Cleveland, consisting of the main and approach piers and caissons. Mr. Grady planned and conducted the work on the Pennsylvania Railroad Bridge over the Maumee River, at Toledo, Ohio, on the extensive ore docks for the Cincinnati, Hamilton and Dayton Railroad Company, and on a large coal dock for the Hocking Valley Railroad Company, both at Toledo.

During the construction of these larger works, he was also in charge of numerous Government and industrial developments on Lake Erie, at Ashtabula, Sandusky, Conneaut, and Lorain, Ohio. At the time of his death, he was a member of the committee of the Cleveland Engineering Society for the straightening of the Cuyahoga River.

In July, 1919, Mr. Grady left the Great Lakes Dredge and Dock Company to become one of the principal partners in the Central Dredging Company, of Cleveland, and just prior to his death, he had organized the Substructure Company of Cleveland.

He was a systematizer of work, and his orderly mental processes were evidenced by the manner in which he planned and directed everything within his jurisdiction. He developed many new and useful methods of construction, and his inventive genius and originality were always at the service of his employers and clients. He was an indefatigable worker, and, although he always had the end in view, nevertheless, he appeared to work for the joy of working.

Mr. Grady was a "self-made man", in the best sense of the word, "self made", but never "self satisfied", for he continued diligent and tireless in making and improving his structure of manhood until his death, which occurred on May 19th, 1921.

On June 8th, 1898, he was married to Miss Alice Archer, of Chicago, who survives him. This partnership was the inspiration of both his business and his private life and the sustaining force which kept him happy and hopeful through the years of his last illness.

His creed and his code were not matters of oral profession, but were evidenced by his daily life and conduct. When once asked to express his rule of life, he replied: "Faithul performance of contract."

Mr. Grady was elected an Associate Member of the American Society of Civil Engineers on December 6th, 1905.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE RELATION BETWEEN DEFLECTIONS AND STRESSES IN ARCH DAMS*

By F. A. Noetzli,† Assoc. M. Am. Soc. C. E.

Synopsis

In this paper there is developed a method of calculating the stresses in arch dams, which result from the deformations and corresponding deflections of the arches, due to water pressure, change of temperature, shrinkage, lateral deformation, swelling, etc., provided the deflections of such arch dams have been measured or have been predetermined in some other way.

This method is based on the following fundamental principle: The deflection of an arch of known dimensions and material can be calculated by the elastic theory, if the stresses due to loads or other influences such as change of temperature, shrinkage, swelling, etc., are known. Consequently, if the deflections of the arch have been measured, the stresses which correspond to such deflections can be calculated in the opposite way.

The stresses which in an arch result from the shortening or lengthening of the arch axis, and the corresponding deflections, may be called "arch deflection stresses" to distinguish them from the direct axial stresses resulting from water pressure.

A few examples are given, which show the importance of the arch deflection stresses in some existing dams. These examples emphasize also the great desirability of measuring the deflections of all arch dams in order to indicate ways and means of designing future similar structures on a more economical and also on a safer basis.

Introduction

Although a great many slender arch dams have been built, which have proved beyond any doubt the great superiority of the arch over any other form of construction, very few tests have been made to determine the actual stresses

* This paper will not be presented for discussion at any meeting of the Society, but written communications on the subject are invited for subsequent publication in *Proceedings*, and with the paper in *Transactions*.

† San Francisco, Cal.

and the factor of safety for such structures under different load and temperature conditions. This is all the more astonishing as the cylinder theory on which the design of most arch dams is based, is known to be defective in a great many ways. A comparison between theoretical and actual stresses, therefore, would have been very desirable, but, apparently, most designers were, or had to be, satisfied if the dams withstood successfully the water pressure. It is a fact that up to the present time no arch dam has failed, but this is hardly an excuse for neglecting the collection of data which would be of the greatest value for future construction.

It is well known that arch dams are statically complicated structures, and any attempt toward a mathematically correct solution of this problem will result either in complicated computations, or it will be futile. With the present knowledge, this problem can be solved by approximations only, and many engineers are still in doubt in regard to the best and safest method of arch dam design. Furthermore, the assumptions on which the design of an arch dam has been based with regard to modulus of elasticity, range of temperature, influence of shrinkage, swelling, lateral expansion, etc., are still more or less uncertain, and can be standardized only by tests and extensive experiments.

These considerations show clearly the great desirability of having deflections and stresses of arch dams measured under different load and temperature conditions. In many other problems, for instance, in hydraulics for the flow of water in pipes, canals, and over weirs, etc., purely theoretical formulas fail, and resort has been had to experiments, whereby coefficients and other data can be determined, to amend the fundamental theory of flow in such a way as to make workable formulas available for all practical cases. In a similar manner, tests and experiments will be necessary to establish a standard method of arch dam design.

Present Theories Used for Arch Dam Design

Ordinary Cylinder Theory.—This is the earliest method of design, and by it an arch dam is considered as a theoretical, free cylinder. For the purpose of design, every arch slice between two horizontal planes is considered as free to move relatively to other similar elementary arches.

This method of design of arch dams neglects entirely, first, that the lowest portions of a dam are practically immovable relatively to the foundation, and that, consequently, no arching can take place, so that the water pressure at the bottom is resisted entirely by the weight of the dam (gravity or cantilever action); and, second, that the highest arches are stressed and forced to deflect, although theoretically they may not have to support any direct water pressure.

Theoretically, therefore, there are serious objections against designing arch dams by the cylinder formula alone. Nevertheless, it is a fact, that no arch dam has failed yet, and this, with the simplicity of the formula itself, is undoubtedly a strong argument in favor of the cylinder theory. However, arch dams of unprecedented magnitude are being contemplated for the near future. Also, there is a tendency to increase the unit stresses, and this increase may be justified only if the stresses in the arches are calculated more accurately than is possible by the simple cylinder theory.

Monolithic Cylinder Theory.—By this method* of design, an arch dam is considered to be a monolithic cylinder, fixed at the base and along the side-hills. This method, although theoretically as correct as can be expected for such a complicated case and for the assumptions made, is applicable only to some special cases, and its use, therefore, is limited.

Theory of Combined Cantilever and Arch Action.—This method† of designing arch dams is based on the assumption of combined cantilever (gravity) and arch action. By simple graphical constructions the division of the water pressure between vertical cantilever and horizontal arches may be obtained in any vertical section of the dam, that is, at the crown of the arch as well as nearer the side-hills. This method permits also of due consideration being given to shrinkage and temperature deformations, rib shortening, swelling, lateral deformation (Poisson's ratio), etc., and the stresses resulting from these various influences can easily be determined and taken care of by reinforcement and a suitable distribution of the dam material.

Stresses in Arch Dams Determined from Deflection Measurements

It is a well recognized fact that a definite load acting on a structure produces a definite deflection, the size of which can be determined accurately if the theory of the structure and the basic assumptions underlying such theory are known. For instance, if an arch bridge is loaded with certain loads the deflections of the bridge due to such loads may be calculated by the theory of the elastic arch. On the other hand, if the deflections have been measured at a certain known temperature, the stresses in the material can be calculated with great accuracy, even if the load itself which produces the deflections is not accurately known.

Exactly the same reasoning applies to the relation between deflections and stresses in arch dams. If the load is known, the stresses and deflections may be calculated. If, on the other hand, the deflections have been measured, the stresses which produce such deflections can be determined by calculation. Deflections due to changes of temperature and those resulting from the shrinkage of concrete, etc., may be treated in a manner similar to that used for deflections due to direct loading.

Stresses which occur in the cantilevers, due to deformations and deflections along such beams in a dam, can be calculated by the well known theory of ordinary cantilever beams.

Stresses in the Horizontal Arch Slices Determined from the Arch Deflections

A number of measurements showing the deflections of arch crowns have been made for the Barren Jack Dam in Australia‡ and for the Salmon Creek Dam in Alaska.§ The deflection curves of these dams aroused considerable

* "Arched Dams", by B. A. Smith, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

† "Gravity and Arch Action in Curved Dams", by Fred A. Noetzli, Assoc. M. Am. Soc. C. E., Pamphlet 20-C, Am. Soc. C. E. (August, 1920).

‡ *Minutes of Proceedings*, Inst. C. E., Vol. CLXXVIII.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 316.

interest among engineers, but there seemed to be no way to derive any benefit from these measurements for the future design and construction of similar structures.

As a matter of fact, these deflection curves are of the greatest interest and significance if interpreted mathematically. The method of combined cantilever and arch action provides a way of interpreting the real meaning and the natural consequence of such deflections, and permits of the calculation of the size of the stresses which were necessary to bend and deflect the structure in the manner shown by the deflection curves.

If the length of an arch is increased or decreased by any forces or influences, the arch crown is moved forward or backward accordingly. This produces bending moments if the span of the arch remains unchanged. Consequently, in addition to the axial compression stresses due to the water pressure, bending stresses also occur in an arch dam. For reservoir full, the arches may be considered to be fixed at the abutments; for reservoir empty, the degree of "fixity" is somewhat uncertain, unless the dam is anchored to the rock foundation by reinforcing steel bars.

Approximate Formulas for the Determination of Arch Deflection Stresses

It has been shown by the writer in a previous paper* that a change, ΔL, in the length, L, of an arch slice, 1 ft. thick produces a horizontal thrust:

$$H = \frac{E\,\Delta L}{\displaystyle\int_{-\frac{l}{2}}^{+\frac{l}{2}} \frac{y^2\,d\,L}{I}} = \frac{45\,E\,I\,\Delta L}{4\,h^2\,l} = 0.94\,\frac{E\,t^3}{h^2\,l}\,\Delta L \ldots\ldots\ldots(1)$$

in which H = horizontal arch thrust acting at the distance of one-third of the rise of the center line measured from the crown;

L = length of center line of arch;

ΔL = change of length, L, due to any cause, such as compression, shrinkage, temperature, etc.;

l = span of arch;

h = rise of arch;

I = moment of inertia of arch $= \frac{1}{12}\,t^3$;

y = ordinate of any point of the arch axis with regard to coordinates through one abutment of the arch;

t = thickness of arch slice;

E = modulus of elasticity.

For the derivation of Equation (1), deformations resulting from bending moments only were considered. This leads to fairly accurate results for thin arches of high rise. For thick and flat arches, however, the effect of direct and shearing stresses becomes very noticeable. The equation for the thrust, H, is then rather complicated. In analyzing a great number of arches of such dimen-

* "Gravity and Arch Action in Curved Dams", Pamphlet 20-C, Am. Soc. C. E. (August, 1920).

sions, as they occur in general in arch dams, the writer has found that by consideration of direct stresses and shear, in addition to bending moments, a fairly accurate value of the total deformation thrust, H, may be obtained by Equation (1a):

$$H = 0.75 \frac{E\, t^3}{h^2\, l} \Delta L \quad \text{.....................(1a)}$$

The writer believes that for the sake of simplicity the use of Equation (1a) is justified in all average cases for which a maximum degree of accuracy is not of primary importance. Adequate means for determining the deformation thrust, H, more accurately are given later.

The crown deflection, D, of an arch is given approximately by the equation:*

$$D = \frac{3}{16} \frac{L}{h} \Delta L \quad \text{.........................(2)}$$

If, in an arch dam, the deflections, D, of the arches have been measured, it is possible to calculate also the corresponding variations, ΔL, of the length of the arch which caused the deflections, D.

Thus, from Equation (2),

$$\Delta L = \frac{16\, h}{3\, L} D \quad \text{.... ....(3)}$$

Introducing Equation (3) in Equation (1),

$$H = 0.75 \frac{E\, t^3}{h^2\, l} \frac{16\, h}{3\, L} D = \frac{4.0\, E\, t^3}{h\, L\, l} D$$

For the arch slices in most arch dams, the length, L, of the arch does not differ greatly from the span, l, and therefore

$$H = 4.0 \frac{E}{L^2} \frac{t^3}{h} D$$

and because $\frac{L^2}{h} = 8.3\, R$ (approximately),

$$H = 0.48 \frac{E}{R} \frac{t^3}{h^2} D \quad \text{.........................(4)}$$

Thus, if the crown deflections, D, of an arch dam have been measured for a number of horizontal arch slices, the horizontal thrust, H, for each one of these arch slices of a vertical thickness of 1 ft. is given in pounds by Equation (4) if all dimensions are measured in feet. H is to be taken as plus in the case of a lengthening of the arch, and minus in case of a shortening.

The thrust, H, which acts at a distance of practically one-third of the rise, h, from the arch crown, produces at any point of the arch a bending moment,

$$M = H \times \text{distance of point from the thrust, } H.$$

At the crown,

$$M = H \times \frac{h}{3} = 0.48 \frac{E}{R} \frac{t^3}{h^2} D \times \frac{h}{3} = 0.16 \frac{E}{R} \frac{t^3}{h} D \quad \text{........(5)}$$

* "Gravity and Arch Action in Curved Dams", Pamphlet 20–C, Am. Soc. C. E. (August, 1920).

The thrust, H, and the moment, M, combined produce the stresses.

$$f_D = \frac{H}{t} \pm \frac{M}{\text{Sec. Mod.}} = 0.48 \frac{E}{R} \frac{t^3}{h^2} \frac{D}{t} \pm \frac{0.16 \frac{t^3}{h} \frac{E}{R} D}{\frac{1}{6} t^2}$$

$$f_D = 0.48\, D \frac{E}{R} \frac{t}{h} \left(\frac{t}{h} \pm 2 \right) \dots\dots\dots\dots\dots\dots (6)$$

in which D is in feet and f_D is pounds per square foot.

If D is measured in inches and all the other dimensions are given in feet, Equation (6) may also be written in the form,

$$f_D = \frac{D}{25} \frac{E}{R} \frac{t}{h} \left(\frac{t}{h} \pm 2 \right) \dots\dots\dots\dots\dots\dots (7)$$

The upper sign in Equations (6) and (7) gives the stresses, f_D, at the intrados, and the lower sign the stresses at the extrados if plus stands for compression and minus for tension. The deflection, D, is thereby to be considered plus for a deflection in an up-stream direction, and minus for a deflection in a down-stream direction.

In a fixed arch, the bending moments at the arch abutments resulting from the thrust, H, are twice as large as at the crown, because the lever arm, for H is twice as long. The stresses are, therefore,

$$f_D = 0.48 \frac{t^3}{h^2} \frac{E}{t\, R} D \pm \frac{-2 \times 0.16 \frac{t^3}{h} \frac{E}{R} D}{\frac{1}{6} t^2}$$

$$f_D = 0.48\, D \frac{E}{R} \frac{t}{h} \left(\frac{t}{h} \pm 4 \right) \dots\dots\dots\dots\dots\dots (8)$$

for D in feet, or

$$f_D = \frac{D}{25} \frac{E}{R} \frac{t}{h} \left(\frac{t}{h} \pm 4 \right) \dots\dots\dots\dots\dots\dots (9)$$

for D in inches.

The upper sign in Equations (8) and (9) gives the stresses, f_D, at the extrados and the lower sign the stresses at the intrados if plus stands for compression and minus for tension. The arch deflection, D, is again to be considered plus for a deflection in an up-stream direction and minus for a deflection in a down-stream direction.

As a general rule, it should be remembered that a lengthening of the arch axis (deflection in an up-stream direction) produces a positive thrust, H, and that a shortening of the arch axis (deflection in a down-stream direction) corresponds to a negative thrust, H. This "deflection thrust", H, which acts at a distance of one-third of the rise of the arch from the crown, therefore, must not be confounded with the axial arch thrust from water pressure.

A deflection of an arch in an up-stream direction (deflection thrust, H, is plus) produces compression at the down-stream side near the crown and at the up-stream side near the abutments. At the same time tension is produced at the up-stream side near the crown and at the down-stream side near the abutments.

A down-stream deflection (deflection thrust, H, is minus) produces tension at the down-stream side near the crown and at the up-stream side near the abutments, and, further, compression at the up-stream side near the crown and at the down-stream side near the arch abutments.

The arch deflection stresses in any horizontal dam slice for which the crown deflection, D, was measured may be calculated therefore by Equation (6) or Equation (7) for a section at the crown and by Equation (8) or Equation (9) for the abutments. These deflection stresses have to be combined with the direct arch compression stresses from water pressure, in order to furnish the maximum stresses for the arch slice under investigation.

Stresses in the Vertical Cantilevers of Arch Dams

It has been shown previously that, particularly at the time of low dam temperature, practically all the water pressure near the base of arch dams is supported by pure cantilever action. In the upper portions of a dam horizontal arch action generally will prevail, but, nevertheless, the cantilever is forced to be bent and deflected by the respective movements of the arches. Such movements obviously introduce stresses of corresponding size in the cantilever. It is possible to calculate the size of these stresses if the deflections have been measured. The method of doing this is based on the well known theory of stresses and corresponding deflections in cantilever beams, and, therefore, will need no further explanation. A few illustrations, however, will be given in some examples discussed subsequently.

The Statical Conditions of Existing Arch Dams

Most of the existing arch dams have been designed and built according to the ordinary cylinder theory. The fact that many of these dams have developed serious cracks shows clearly that although pure arch action has been assumed in the design, temperature and shrinkage stresses are able at certain times to destroy such action entirely, because no arching can occur through open cracks.

If a reservoir formed by an unreinforced arch dam is empty, it has been shown by experience that in most such dams the opening of the vertical construction joints or other cracks is due to the shrinkage of the concrete during the setting and hardening of the cement and also to a decrease of temperature during the cold months of the year. Such a structure, before any water pressure acts on it, consists under those circumstances of a series of free vertical cantilevers.

Suppose, now, that the water rises slowly in the reservoir. At the beginning, its pressure will be supported entirely by the vertical cantilevers, because the contraction joints and other cracks are open, and it is evident that no arch stresses can be transmitted across open cracks. As soon as the cantilevers have deflected, under the pressure of the rising water, so far in a down-stream direction that all the vertical joints and cracks in the dam are closed, arching may occur and arch pressure is transmitted sidewise to the abutments. In many existing arch dams, the water had to rise more than one-half the height of the dam before the vertical joints and cracks were closed. For instance,

for the Salmon Creek Dam in Alaska, which is a constant-angle arch dam, 168 ft. high, the water had to rise more than 100 ft. in the reservoir to be sufficient to keep the contraction joints closed.* In other words, when the water rose for the first time in the reservoir, this dam acted by pure vertical cantilever action until the water had reached a depth of slightly more than 100 ft. Then the contraction joints closed tightly under this pressure, and the arch stresses could aid in supporting the additional pressure when the water rose still higher.

Similar conditions exist for many other slender arch dams, and this is one of the reasons why such structures can be considered with very close approximation as a combination of vertical cantilevers and horizontal arches. Also, for structures which have not developed open vertical cracks visible to the eye, it is well known that contraction of the concrete occurs under the influence of shrinkage and low temperature just as it does in any other concrete structure. Good concrete may carry tension of from 100 to 200 lb. per sq. in., or develop a great number of hair cracks, particularly in reinforced structures. Such conditions again decidedly favor initial vertical cantilevering in the lower portions of such a dam, until either all these hair cracks are closed or the shrinkage tension in the arches is relieved so that actual arch compression can occur. To close the vertical hair cracks or to relieve such shrinkage tension as may exist in a horizontal direction in a dam, however, the vertical cantilever has first to be deflected a corresponding amount. This also shows clearly that in such a dam the water pressure near the base, of necessity, has to be supported almost entirely by vertical cantilever action.

It has been proposed to force grout into the contraction joints "and put initial axial compression into the whole structure, thereby making it act like a solid arch."† Such grouting if it could be extended uniformly all over a crack would be of great advantage indeed. However, serious objections have been voiced against the grouting of contraction joints. Such joints, or other cracks, as is well known, may be $\frac{1}{4}$ or $\frac{1}{8}$ in. wide in the upper parts of a dam, but diminish to nil at or slightly above the foundation. In the lower 50 ft. of a dam of a height of, say, 150 ft., the width of such open joints is therefore on an average probably less than $\frac{1}{32}$ or $\frac{1}{64}$ in., and it is more than doubtful whether such fine cracks can be filled evenly by cement grout which does not contain an excessive quantity of water. There exists, therefore, the great danger that large spaces in the joints will remain unfilled and that all the arch thrust due to the pressure of the rising water will be concentrated in those areas which were reached by the grout. This will often result, not only in an uneven distribution of the arch stresses, but also in large eccentricities of the arch thrust, thus producing heavy bending moments which may ultimately become a serious menace to the safety of such a dam.

On the other hand, if only a part of the cross-section of a dam is brought to carry the arch thrust, what help is to be expected from those portions of the structure which lie opposite the unfilled spaces of the contraction joints?

* "Improving Arch Action in Arch Dams", by L. R. Jorgensen, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 320.

† *Loc. cit.*, p. 323.

Here is the old problem of the case of a chain with some very strong and some rather weak links. If, however, it is recognized that the difficulties of filling contraction joints evenly with grout are practically unsurmountable, how should it be possible, as is claimed, to fill these joints under such a pressure as to force the arch crown in an up-stream direction and to put shear on the foundation in the opposite direction to that due to the water load?* The pressure of the liquid grout can in no case be greater than the pressure due to the static head of this liquid, as in the case of any other vessel which is open at the top. The additional pressure shown by the grout pump is simply friction head of the grout flowing in the pipes and through the narrow cracks in the dam. Besides, there remain admittedly free air spaces in the joints, and it is difficult to imagine such air bubbles to be under considerable pressure while they could easily escape toward the open top.

Furthermore, an actual initial stress in the arches and a corresponding movement in the up-stream direction could only occur if the last contraction joint to be filled was widened to make possible the lengthening of the arch corresponding to the supposedly up-stream deflection. For instance, if a dam of the dimensions of the Salmon Creek Dam, referred to previously, should be grouted under such a pressure as to force the crown at the crest for, say, $\frac{1}{2}$ in., in an up-stream direction, the contraction joint under pressure would have to be widened by more than $\frac{1}{2}$ in. Of course, this would release at once the caulking material with which the down-stream side of the contraction joint was closed, and the grout would readily flow out.

It was deemed advisable to discuss somewhat at length the question of grouting the contraction joints of arch dams for the reason that important structures of this kind have been grouted in the past, and similar work is contemplated for the near future. For reasons already given, the grouting of contraction joints and other cracks is considered not only ineffective, as far as putting initial stresses into the arches is concerned, but, in certain cases, it may seriously affect the strength of an arch dam in which for one reason or another the joints have not been filled evenly. On the other hand, the construction joints of a straight gravity dam may be grouted with excellent results in order to make such joints water-tight.

A procedure which is undoubtedly much safer than grouting consists of building or closing the arch during the colder season of the year. If this is not possible, for one reason or another, vertical slots of suitable width should be left during the construction and later filled during the colder season. In both cases, however, proper judgment must be used with regard to the closing temperature in order that the arches and the cantilever are not over-stressed at the time the arch temperature rises to the seasonal maximum and when the reservoir is empty.

Examples

Practically all existing arch dams, with the exception of the Wooling Dam in Australia, have been designed and built according to the ordinary cylinder theory. As far as the writer knows, the Wooling Dam is the only structure

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 326.

of its kind designed as a monolithic cylinder fixed at the base. The deflections of this dam were measured and a close coincidence between calculated and measured deflections was found, so that the correctness of the monolithic cylinder theory as applied to this dam seems to be established beyond any doubt.

The writer's graphical method of investigating arch dams by considering such structures as acting like a combination of vertical cantilevers and horizontal arches has been proved* to give results which check almost exactly with those obtained by the mathematically correct method given by B. A. Smith, M. Am. Soc. C. E.† Thus, also, the soundness of the theory of combined cantilever and arch action is established, at least with this example. It may be of interest, therefore, to apply those principles to some other dams for which deflections have been measured.

The Barren Jack Dam in Australia.—This is an arch dam of a maximum height of 42 ft. above bed-rock, the radius of the up-stream face being 80 ft. The dam is 5.0 ft. thick at the base and 2.0 ft. thick at the crest. Fig. 1 shows a cross-section and various deflection curves for the arch crown under different load and temperature conditions.

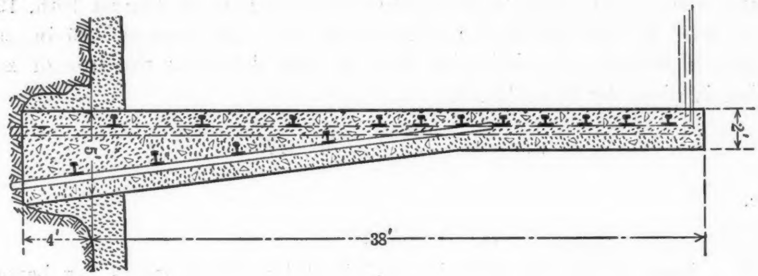
A striking feature of the deflection lines is the comparatively large angle at the foundation between the deflection curves and the line of zero deflection. Undoubtedly, the curves drawn through the various observed points have their origin at one common point at the foundation base. This point of origin, undoubtedly, is practically the same for any load and corresponding deflection curves for the simple reason that the bed-rock prevents any appreciable movement of the base of the dam. Elastic deformations are in some ways proportional to the unit stresses in the material, and the unit stresses in the large masses of bed-rock, resulting from the shearing and bending forces exerted by the dam on the rock, are so small at a short distance below the plane of contact between dam and bed-rock that the corresponding elastic deformations of the bed-rock itself are negligible. Any investigation of the bed-rock deformations at the base of any of the existing arch dams will prove this beyond any doubt.

Consequently, it follows that if the bed-rock is not deformed elastically or in any other manner for any appreciable amount, the dam may be considered to be rigidly fixed at the base unless cracks should have developed either between the masonry and the rock or in the masonry itself at the time when the dam came under pressure and was deflected more than is elastically possible.

Consider, now, a vertical dam slice at the crown of the arches where the deflections were measured. If such a cantilever was rigidly fixed at the base either due to its own weight or by special anchorage to the bed-rock, any deflection line would have the original line of zero deflection as a tangent at the origin at the base. The deflection curves of the Barren Jack Dam show a marked angle at the origin between the line of zero deflection and the

* Discussion on "Gravity and Arch Action in Curved Dams", by William Cain, M. Am. Soc. C. E., Pamphlet 20-C-2, Am. Soc. C. E., (December, 1920), p. 1.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919-20), p. 2027.


5'
2'
4'
38'

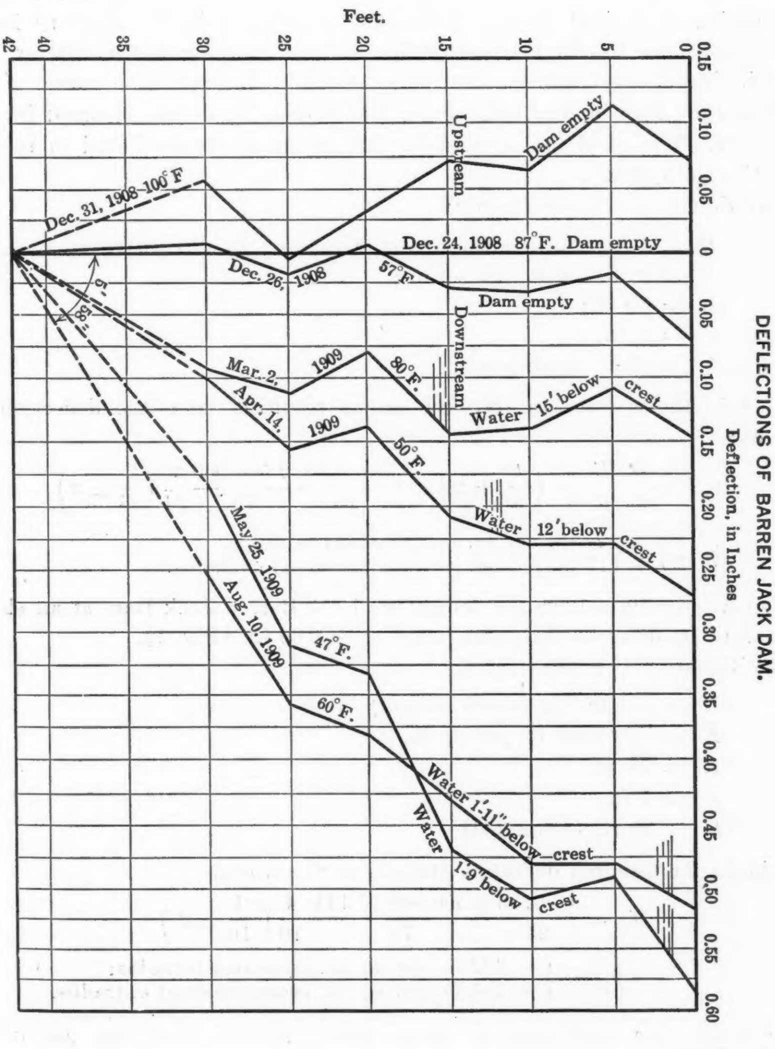

DEFLECTIONS OF BARREN JACK DAM.
Deflection, in Inches
Feet.
42
40
35
30
25
20
15
10
5
0
0.15
0.10
0.05
0
0.05
0.10
0.15
0.20
0.25
0.30
0.35
0.40
0.45
0.50
0.55
0.60
Upstream
Downstream
Dec. 31, 1908-100° F
Dam empty
Dec. 24, 1908 87° F. Dam empty
Dec. 26, 1908
57° F.
Dam empty
Mar. 2, 1909
80° F.
Water 15' below crest
Apr. 14, 1909
50° F.
Water 12' below crest
May 25, 1909
47° F.
Aug. 10, 1909
60° F.
Water 1'-11" below crest
Water 1'-9" below crest
5'
38'

FIG. 1.

deflection curve itself. For instance, the measurements of August 10th, 1909 (Fig. 1), gave the deflection of a point 12 ft. above the base as 0.25 in., and the angle, α, between the deflection line on that date and the line of zero deflection is given by the relation,

$$\tan \alpha = \frac{0.25}{12 \times 12} = 0.00173$$

whence,

$$\alpha = 0° \ 05' \ 58''$$

Such a large angle can only be explained by deformations far beyond elastic ones, or, in other words, by open cracks either between the dam and the foundation or a short distance above the foundation in the masonry itself. That such open cracks may occur due to over-stressing the cantilever, has been indicated previously by other calculations.* The deflection curves of the Barren Jack Dam undoubtedly prove that such a break has occurred in this structure. The maximum arch deflection at the crest was found to be 0.58 in. (May 25th, 1909).

By Equation (7), substituting

$D = -0.58$ in. (down stream, therefore, D is negative);
$E = 2\,500\,000$ lb. per sq. in.;
$R = 79$ ft.;
$t = 2$ ft.;
$h = 24$ ft. (estimated)

we obtain the stresses in the arch crown resulting from the deflection as follows:

$$f_D = \frac{D}{25} \frac{E}{R} \frac{t}{h} \left(\frac{t}{h} \pm 2 \right) = \frac{-0.58}{25} \frac{2\,500\,000}{79} \frac{2}{24} \left(\frac{2}{24} \pm 2 \right)$$

$$f_D = \begin{cases} -127 \text{ lb. per sq. in. (tension) intrados.} \\ +117 \text{ lb. per sq. in. (compression) extrados.} \end{cases}$$

On August 10th, 1909, the deflection of the Barren Jack Dam at an elevation of 17 ft. above the base was found to be 0.36 in. (Fig. 1).

By Equation (7), substituting

$D = -0.36$ in. (down stream)
$E = 2\,500\,000$ lb. per sq. in.
$R = 78$ ft.
$t = 4$ ft.
$h = 10$ ft. (estimated)

we obtain for the arch deflection stresses at the crown,

$$f_D = \frac{-0.36}{25} \frac{2\,500\,000 \times 144}{78} \frac{4}{10} \left(\frac{4}{10} \pm 2 \right)$$

$$f_D = \begin{cases} -442 \text{ lb. per sq. in. (tension) intrados;} \\ +295 \text{ lb. per sq. in. (compression) extrados.} \end{cases}$$

* "Gravity and Arch Action in Curved Dams", Pamphlet 20-C, Am. Soc. C. E., (August, 1920).

If the same horizontal arch slice had to support all the water pressure, as is generally assumed when only the ordinary cylinder theory is considered, the compression stresses in the arch due to the water pressure would be:

$$f_w = \frac{P\,R_u}{t} = \frac{23 \times 62.5 \times 80}{4 \times 144} = 200 \text{ lb. per sq. in.}$$

where P = water pressure; and,
R_u = up-stream radius

The total stresses in this arch slice at the crown, therefore, would be,

$$f_D = \begin{cases} -\ 242 \text{ lb. per sq. in. (tension) intrados;} \\ +\ 495 \text{ lb. per sq. in. (compression) extrados.} \end{cases}$$

Due to the cantilever action which undoubtedly helps in supporting the water pressure at this elevation, the direct arch stresses are reduced somewhat. The correct values could be obtained by applying the method of combined cantilever action and horizontal arching.

In a similar manner, as shown previously, the arch deflection stresses and the maximum stresses may be determined at other elevations for any horizontal arch slice and for any characteristic load and deflection in an up-stream or down-stream direction.

It is needless to say that in a scientifically designed arch dam all excessive tension stresses should be taken care of by sufficient steel reinforcement to prevent the occurrence of tension cracks.

The Salmon Creek Dam in Alaska.—The Salmon Creek Dam, near Juneau, Alaska, is the first dam designed and built according to the modern principles of the constant-angle or varying radius types. The deflections of this structure were measured on a number of days when the water in the reservoir stood at different elevations and when the dam temperature had varied considerably from the normal.

The deflection curves of this dam have been published and discussed by L. R. Jorgensen, M. Am. Soc. C. E., in his paper, entitled "Improving Arch Action in Arch Dams".* A further interpretation of these deflections, based on the method of combined vertical cantilever and horizontal arch action, will be given by the writer. It is believed that certain phenomena occurring in the deflections of this dam and not clearly recognized heretofore may be more easily understood in this way. For the sake of convenience, the deflection curves of the Salmon Creek Dam are reproduced in Fig. 2.

The dotted line, *O-A-B,* in Fig. 2, shows the theoretical deflections of the arches for full water pressure and by neglecting cantilever action (simple-cylinder theory), it is evident that the theoretical and the actual deflections do not coincide with each other.

A slight modification in the measured deflection curves has been made in Fig. 2, in that the dotted lower portions of the lines have been brought to a common origin. For reasons explained in more detail in connection with the example of the Barren Jack Dam (page 270), it is believed that the large masses of solid bed-rock into which the base of the dam was keyed, did not deform appreciably under the stresses exerted by the dam, at least not so much as to

* *Transactions,* Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 316.

enable such elastic down-stream deformation to be measured and to be shown on the diagram.

The most striking feature of the deflection lines of this dam is again, as in the case of the Barren Jack Dam, the large angle at the base between the deflection curves and the line of zero deflection. This phenomenon is expressed most decidedly for the deflections measured on October 27th, 1915 (Curve No. 7). At Elevation 1035 (28 ft. above the base), the deflection measured on that day was found to be 0.375 in.

The angle, α, is given by the relation,

$$\tan \alpha = \frac{0.375}{28 \times 12} = 0.000111$$

and,

$$\alpha = 0° \; 03' \; 50''$$

A cantilever fixed at the base and deflected elastically by loads above could not form such a large angle at the base, but would have the line of zero deflections as a tangent, approximately as shown by a typical cantilever deflection curve in Fig. 2.

In view of the fact that the deflections of the Salmon Creek Dam show clearly a large angle between the deflection curves and the line of zero deflection, it is evident that one or a series of cracks have opened in the cantilever on the up-stream side of this structure. These cracks may have occurred either between the dam and the bed-rock or in the masonry itself a short distance above the bed-rock, wherever the least tensile strength existed. Such cracks, of course, will extend only for a comparatively short distance into the dam body, and the water-tightness is insured sufficiently by the increased compression stresses near the down-stream side.

A further proof that this dam has developed horizontal cracks at the up-stream side is shown by the following calculation:

Consider a vertical cantilever slice at the center of the dam, where the deflections have been measured. The measurements of October 27th, 1915 (Curve No. 7), shows the deflection of this cantilever to have been 0.375 in. at a distance of 28 ft. above the base. Neglect, now, for the purpose of this investigation, the portions of the vertical cantilever above Elevation 28. There remains, then, a cantilever of, say, a lateral width of 1 ft. and a depth equal to the thickness of the dam, or, on an average, about 42 ft. in this particular case. The length of this cantilever is 28 ft. and the deflection of the upper end, that is, 28 ft. from the fixed end at the base, has been measured at 0.375 in. (Fig. 3).

In order to deflect such a cantilever for 0.375 in., a uniformly distributed load of about

$$W = 20\,000\,000 \text{ lb.}$$

would be necessary. The bending moment at the fixed end would then be,

$$M = 20\,000\,000 \times \frac{28}{2} = 280\,000\,000 \text{ ft-lb.},$$

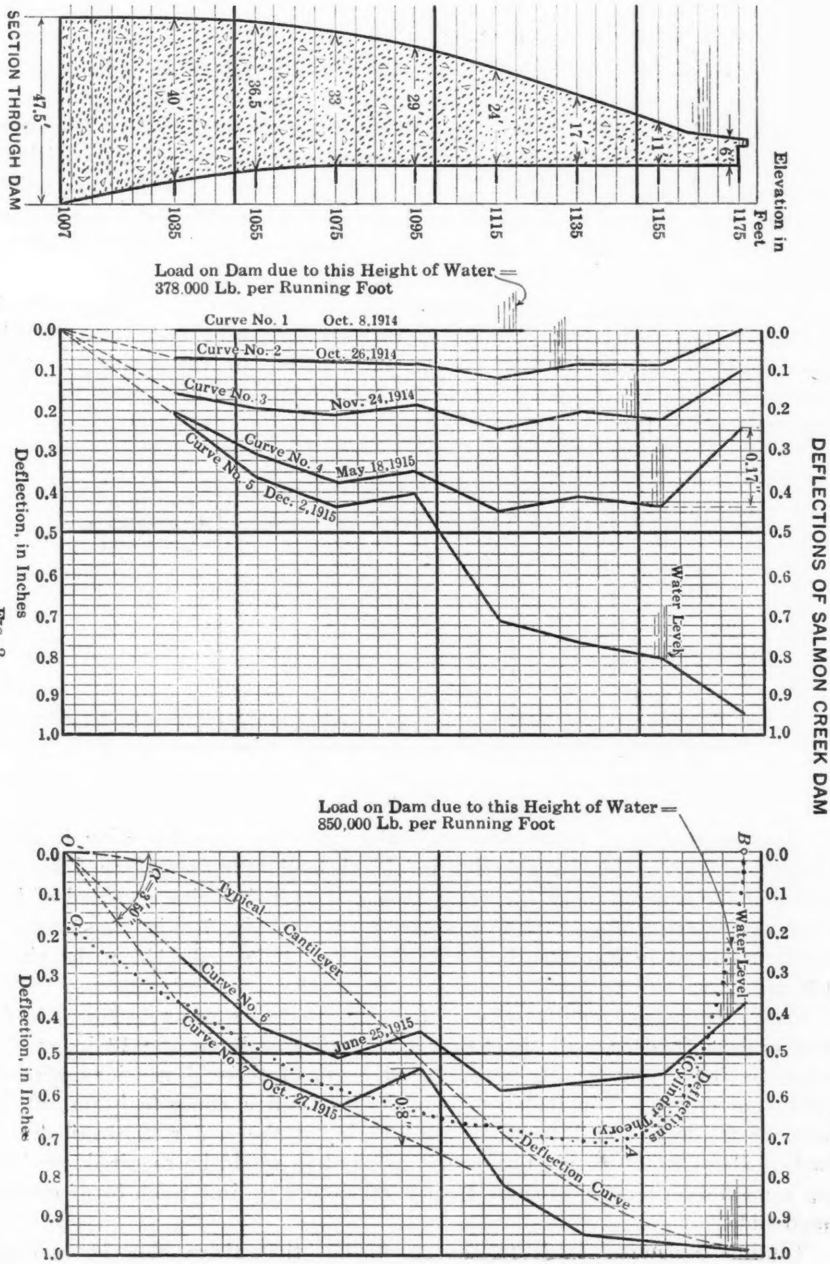

DEFLECTIONS OF SALMON CREEK DAM
SECTION THROUGH DAM
Elevation in Feet
47.5′
40′
36.5′
33′
29′
24′
17′
11′
6′
1007
1035
1055
1075
1095
1115
1135
1155
1175
Load on Dam due to this Height of Water = 378,000 Lb. per Running Foot
Curve No. 1 Oct. 8, 1914
Curve No. 2 Oct. 26, 1914
Curve No. 3 Nov. 24, 1914
Curve No. 4 May 18, 1915
Curve No. 5 Dec. 2, 1915
0.17″
Water Level
Deflection, in Inches
Load on Dam due to this Height of Water = 850,000 Lb. per Running Foot
Typical Cantilever
Curve No. 6 June 25, 1915
Curve No. 7 Oct. 27, 1915
Deflection Curve
Deflections (Cylinder Theory)
Water Level
0.8″
O
O′
A
B
Deflection, in Inches

FIG. 2.

producing stresses of,

$$f = \pm \frac{M}{\text{Sec. Mod.}} = \pm \frac{280\,000\,000}{\frac{1}{6} \times 1 \times 42^2} = \pm\ 950\,000 \text{ lb. per sq. ft.}$$

which, of course, is impossible. Even after consideration of the weight of the cantilever, and all other forces and reactions transmitted from the upper portions of the dam to the short cantilever under investigation, it is clearly seen that such a short and deep cantilever cannot deflect for 0.375 in. without cracking or being partly lifted from the "fixed" end. It is also impossible that the well-seasoned concrete at the base should have "flowed" so much as to permit such a large deflection before the limit of tensile strength of the concrete was reached at the up-stream side.

These mathematical interpretations of the deflections of the Salmon Creek Dam seem to be conclusive proof that this structure also broke away partly from the base and developed horizontal cracks at the up-stream side at a time when the water in the reservoir had risen toward the crest and the dam temperature was low.

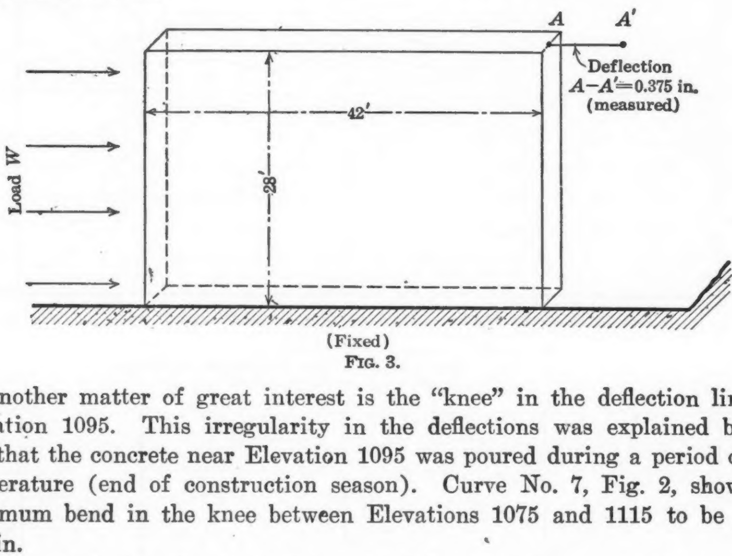


FIG. 3.

Another matter of great interest is the "knee" in the deflection lines at Elevation 1095. This irregularity in the deflections was explained by the fact that the concrete near Elevation 1095 was poured during a period of low temperature (end of construction season). Curve No. 7, Fig. 2, shows the maximum bend in the knee between Elevations 1075 and 1115 to be about 0.18 in.

Any calculations for determining the deflection of a concrete beam, 40 ft. long and 28 ft. deep, will show that a deflection as large as 0.18 in. at the center is impossible for an unreinforced beam of these dimensions without producing cracks. For instance, in order to bend a concrete beam, 40 ft. long, 28 ft. deep, and 1 ft. wide, for 0.18 in. at the center, theoretically, a load of about 9 500 000 lb., uniformly distributed, would be required, producing a maximum tension of more than 2 500 lb. per sq. in. Such is, of course, impossible.

Similar reasoning leads to the conclusion that there may be another horizontal crack in the dam near Elevation 1155. The measurements made on May 18th, 1915 (Curve No. 4, Fig. 2), show that, below Elevation 1155,

the deflection line for about 40 ft. was nearly vertical and that, at the crest, the dam deflected about 0.17 in. relatively to Elevation 1155. A short calculation for the cantilever portion above Elevation 1155 shows that, even under conservative assumptions, this cantilever, 20 ft. long, with an average thickness of about 8 ft., would have to sustain tensile stresses of more than 900 lb. per sq. in., in order to allow at the free end a deflection of 0.17 in. This seems to indicate that probably another horizontal break has occurred at or near Elevation 1155, as stated.

The deflection curves of the Salmon Creek Dam therefore lead to the conclusion that in this structure the vertical cantilever is broken at three different elevations. The breaks occurred most probably in horizontal construction joints where new concrete was poured on top of the old, or where the accumulation of laitence had reduced the tensile strength. Although the deflection curves clearly indicate the presence of cracks, nothing can be predicted, of course, with regard to their number, width, and extension, without deflection measurements being taken at other points of the dam. It may be argued that the concrete of the dam "flowed" in excess of the elastic deformations and thus prevented the occurrence of open cracks. However, while it may "flow" somewhat under high compression stresses, it hardly did so to a marked degree before the limit of the tensile strength, say, 150 to 200 lb. per sq. in., of the concrete of the structure under consideration, was reached. It is now a matter of speculation as to what degree such cracks may affect the safety of the structure and whether there is any danger of gradual disintegration under the occurring reversible stresses.

When the water in the reservoir stands, for instance, at Elevation 1095, where a horizontal crack apparently exists, the cantilever and, therefore, the whole dam has the tendency to be deflected somewhat in a down-stream direction. At the same time, the arch ring from Elevation 1095 upward may be forced in an up-stream direction, say, due to a rise in temperature. Thus, shearing stresses of considerable magnitude occur in the plane of the crack, and this fact may lead to a movement and to progressive deterioration, if the climatic conditions are such as to favor deflections in opposite directions.

It may now be of interest to calculate also the "arch deflection stresses" in the horizontal dam slices.

The maximum deflection of the highest arch, as measured on October 27th, 1915, was 0.98 in. (Curve No. 7, Fig. 2). As a matter of fact, it must have been somewhat larger, because the assumed line of zero deflection (Curve No. 1) includes already the deflection necessary to close the open cracks in the dam.*

By Equation (7), substituting

$D = -0.98$ in. (down stream);

$E = 2\,500\,000$ lb. per sq. in.;

$R = 325$ ft.;†

$t = 6$ ft.;†

$h = 150$ ft.;†

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 320.

† These dimensions are taken from Plate XIII, *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 709.

we obtain the stresses at the arch crown resulting from the deflection, as follows:

$$f_D = \frac{-0.98}{25} \times \frac{2\ 500\ 000 \times 144}{335} \times \frac{6}{150}\left(\frac{6}{150} \pm 2\right)$$

$$f_D = \begin{cases} -24 \text{ lb. per sq. in. (tension) intrados;} \\ +23 \text{ lb. per sq. in. (compression) extrados.} \end{cases}$$

On the same day, the deflection of the dam was 0.82 in. at an elevation of 108 ft. above the base (Curve No. 7, Fig. 2). At this elevation,

$$R = 282 \text{ ft.};$$
$$t = 24 \text{ ft.};$$
$$h = 76 \text{ ft.}$$

Then, by Equation (7), the arch deflection stresses at the crown of this arch slice are,

$$f_D = \frac{0.82}{25} \times \frac{2\ 500\ 000 \times 144}{282} \times \frac{24}{76}\left(\frac{24}{76} \pm 2\right)$$

$$f_D = \begin{cases} -213 \text{ lb. per sq. in (intrados) tension;} \\ +154 \text{ lb. per sq. in. (extrados) compression.} \end{cases}$$

The arch deflection stresses at the abutments are found by Equation (9), as follows:

$$f_D = \frac{0.82}{25} \times \frac{2\ 500\ 000 \times 144}{282} \times \frac{24}{76} \times \left(\frac{24}{76} \mp 4\right)$$

$$f_D = \begin{cases} -396 \text{ lb. per sq. in. (extrados) tension;} \\ +338 \text{ lb. per sq. in. (intrados) compression.} \end{cases}$$

These arch deflection stresses which, as must be remembered, are due entirely to the bending of the arch while it was being shortened as the result of axial compression, drop of temperature, shrinkage, etc., have to be combined with the stresses due to the pressure of the water. If all this pressure was carried by horizontal arching (ordinary cylinder theory), the stresses from water pressure in the arch investigated would be:

$$f_w = \frac{58 \times 62.5 \times 294}{24} = +307 \text{ lb. per sq. in. (compression).}$$

This stress combined with the arch deflection stresses gives the maximum unit stresses in this arch slice, as follows:

At the crown..... $\begin{cases} \text{Intrados } f_{max.} = +\ \ 94 \text{ lb. per sq. in. (compression);} \\ \text{Extrados } f_{max.} = +461 \text{ lb. per sq. in. (compression).} \end{cases}$

At the abutments. $\begin{cases} \text{Extrados } f_{max.} = -\ \ 89 \text{ lb. per sq. in. (tension);} \\ \text{Intrados } f_{max.} = +645 \text{ lb. per sq. in. (compression).} \end{cases}$

In this manner, the arch deflection stresses and the true maximum stresses in the arching parts of this dam may be determined at any other elevation and for any load and temperature condition for which deflection measurements are available or may be made any time in the future.

The calculations point to a maximum compression at the intrados near the abutments of slightly more than 600 lb. per sq. in. It is possible that under such high unit stresses the green concrete "flowed" somewhat so that a readjustment took place, which reduced the theoretical unit stresses both

on the tension and on the compression side. Such a "flow" could be best detected by continuous deflection measurements at various critical places on the dam. Furthermore, it has to be considered that whenever, in an unreinforced arch dam, cracks have opened on the tension side, the theoretical maximum compression stresses are also reduced, because the resultant of the compression stresses in such a case has to equal the compression resulting from the external loads.

More Accurate Formulas for the Determination of the Arch Deflection Stresses

As previously mentioned, Equations (6), (7), (8), and (9), are only approximations, their main advantage being their simplicity. Greater accuracy is obtained by taking into consideration, in a theoretically correct manner, also the deformations due to the axial stresses and the shearing stresses, besides those resulting from bending moments.

The elastic theory of arches, as given in most modern treatises on arch design, shows the horizontal arch thrust, H_f, which is due to rib-shortening resulting from loads applied to the arch, to be:

$$H_f = -\frac{H_a \int \frac{d\,L}{A}}{\int y^2 \frac{d\,L}{I} + \int \cos^2 \alpha \frac{d\,L}{A} + 3 \int \sin^2 \alpha \frac{d\,L}{A}} \quad \ldots\ldots (10)$$

whereby, in the case of a horizontal slice of an arch dam, 1 ft. thick:

H_f = arch thrust resulting from rib-shortening;
H_a = arch thrust due to the water pressure;
$d\,L$ = length of an arch element;
A = cross-section of arch (= thickness, t, of arch slice);
y = ordinate of arch elements, $d\,L$, with regard to a horizontal X-axis drawn through the center of gravity of the whole arch;
α = angle of inclination between arch element and horizontal X-axis; and
I = moment of inertia of cross-section of arch $\left(I = \frac{1}{12} t^3\right)$.

According to Hooke's law, H_a has the value:

$$H_a = \frac{\Delta\,L}{L} E\,t$$

where E equals the modulus of elasticity and $\Delta\,L$ equals the total shortening of the arch axis. Further, there is,

$$\int \frac{d\,L}{A} = \frac{L}{t}$$

The numerator of Equation (10) is, therefore, $E\,\Delta\,L$.

The first term in the denominator of Equation (10), for H_f, that is, $\int y^2 \frac{d\,L}{I}$, represents the influence from bending moments; the second term,

$\int \cos^2 \alpha \frac{d\,L}{A}$, gives the influence of the direct stresses; and the third term, $3 \int \sin^2 \alpha \frac{d\,L}{A}$, represents the influence from shearing stresses.

The main term in the denominator of Equation (10) is $\int y^2 \frac{d\,L}{I}$, and, as shown in the writer's paper "Gravity and Arch Action in Curved Dams",* this equals very nearly $\frac{4\,h^2\,l}{45\,I}$. Consequently, H_f has approximately the value,

$$H_f = 0.94 \frac{E\,t^3}{h^2\,l} \Delta L$$

which is the same as that obtained in Equation (1).

In order to obtain a more accurate value of H_f, we will compare its exact value (Equation (10)) with the approximate value given by Equation (19 *a*) of the writer's paper on "Gravity and Arch Action in Curved Dams"*. Equation (10) may be written:

$$H_f = -\frac{H_a \int \frac{d\,L}{t}}{\int y^2 \frac{d\,L}{I} + \int \cos^2 \alpha \frac{d\,L}{t} + 3 \int \sin^2 \alpha \frac{d\,L}{t}} \quad \ldots\ldots (11)$$

Equation (19 *a*) of the writer's paper on "Gravity and Arch Action in Curved Dams", reads:

$$H_f = -0.94\, f'_c \frac{t^3}{h^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

in which f'_c equals the axial stress due to water pressure. The thrust, H_a, in Equation (11) equals $f'_c\,t$, and the whole numerator, therefore, has the value, $f'_c\,L$.

In order to make H_f in Equation (12) equal to the theoretically correct value of H_f in Equation (11), we simply have to introduce in Equation (12), instead of the approximate constant value, 0.94, such a factor, k_f, that Equation (12) furnishes the same values for H_f as Equation (11).

The factor, k_f, may be calculated from the relation:

$$H_f = -\frac{f'_c\,L}{\int y^2 \frac{d\,L}{I} + \int \cos^2 \alpha \frac{d\,L}{t} + 3 \int \sin^2 \alpha \frac{d\,L}{t}} = -k_f\, f'_c \frac{t^3}{h^2}$$

from which,

$$k_f = \frac{h^2\,L}{t^3 \left(\int y^2 \frac{d\,L}{I} + \int \cos^2 \alpha \frac{d\,L}{t} + 3 \int \sin^2 \alpha \frac{d\,L}{t} \right)} \quad \ldots\ldots (13)$$

The values, k_f, have been calculated for arches with central angles of from 60 to 180° and for various proportions of $\left(\frac{t}{h}\right)$, which are plotted in Fig. 4.

* Pamphlet 20–C, Am. Soc. C. E. (August, 1920).

The arch thrust, H_f, which is due to rib-shortening, may be calculated therefore for any arch slice of an arch dam from the following equation:

$$H_f = -k_f f'_c \frac{t^3}{h^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

whereby k_f may be taken from Fig. 4 for the different central angles of the arches and the various proportions of $\left(\frac{t}{h}\right)$.

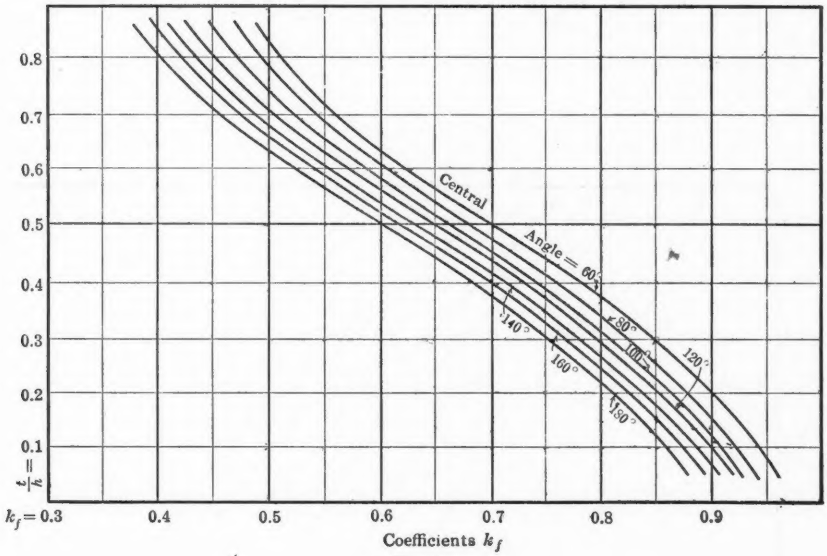


FIG. 4.

The arch thrust, H_f, which acts, as should be remembered, at a distance of approximately one-third of the rise of the arch from the crown, produces bending moments and stresses in the arch. For instance, at the crown the stresses are:

$$f_c = \frac{H_f}{t} \pm \frac{H_f \frac{h}{3}}{\text{Sec. Mod.}}$$

$$f_c = -k_f f'_c \frac{t}{h}\left(\frac{t}{h} \pm 2\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

in which f_c is pounds per square foot; and at the abutments:

$$f_c = \frac{H_f}{t} \pm \frac{H_f \frac{2}{3} h}{\text{Sec. Mod.}}$$

$$f_c = -k_f f'_c \frac{t}{h}\left(\frac{t}{h} \pm 4\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

The deformations resulting from temperature and shrinkage have to be treated separately from those due to rib-shortening.

For temperature deformations, the theory of the elastic arch gives the horizontal thrust:

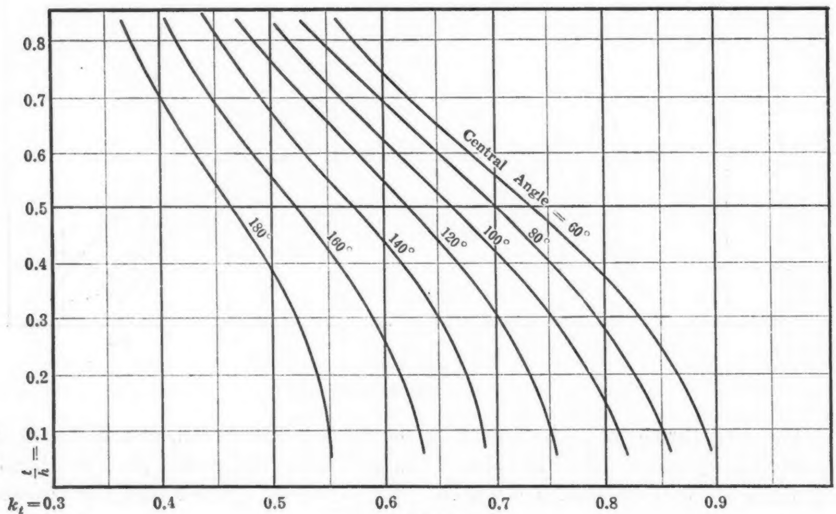
$$H_t = \frac{c\ T\ l}{\int y^2 \frac{d\ L}{I} + \int \frac{d\ L}{A}} \quad \text{.....................} (17)$$

in which,

c = coefficient of expansion;
T = change of arch temperature;
l = span of arch.

In a manner similar to Equation (14) for H_f, Equation (17) for H_t, may be transformed into:

$$H_t = k_t\ E\ c\ T \frac{t^3}{h^2} \quad \text{.......................} (18)$$



Coefficients k_t

FIG. 5.

The numerical values for k_t are calculated and plotted in Fig. 5 for different central angles between 60 and 180° and for various values of $\frac{t}{h}$

The stresses in the arch due to H_t are, at the crown,

$$f_c = k_t\ E\ c\ T \frac{t}{h} \left(\frac{t}{h} \pm 2 \right) \quad \text{...................} (19)$$

in which f_c is pounds per square foot; and at the abutments,

$$f_c = k_t\ E\ c\ T \frac{t}{h} \left(\frac{t}{h} \pm 4 \right) \quad \text{...................} (20)$$

If the deflections, D, of an arch dam have been measured, and it is desired to know the resulting arch deflection stresses, the procedure is as

follows: Insert in Equation (14) for f'_c (axial stress due to water pressure), its value (Hooke's law):

$$f'_c = \frac{\Delta L}{L} E$$

Then,

$$H_f = k_f \frac{E t^3}{h^2 L} \Delta L$$

Introduce for ΔL its value according to Equation (3),

$$\Delta L = \frac{16 h}{3 L} D$$

The deformation stresses in the arch may then be obtained in a manner similar to that previously shown. At the crown,

$$f_D = 0.64 k_f D \frac{E}{R} \frac{t}{h} \left(\frac{t}{h} \pm 2 \right) \qquad (21)$$

in which D is in feet, and f_D is pounds per square foot; and at the abutments,

$$f_D = 0.64 k_f D \frac{E}{R} \frac{t}{h} \left(\frac{t}{h} \pm 4 \right) \qquad (22)$$

The upper sign in Equation (21) gives the stresses, f_D, at the intrados and the lower sign at the extrados, and opposite for Equation (22). Compression is shown by plus and tension by minus. The deflection, D, is to be considered plus for a deflection in an up-stream direction and minus for a downstream deflection.

In Equations (21) and (22) the coefficient, k_f, is to be taken from the curves shown on Fig. 4 if the deflections result mainly from water pressure. If the arch deflection can be traced mainly to temperature or shrinkage deformations, the coefficients, k_t, taken from the curves shown on Fig. 5, should be used instead. If water pressure and temperature together produced the measured deflection, a coefficient has to be interpolated between k_f and k_t according to the relative importance of water pressure or temperature.

Conclusions

The results of the investigations given in this paper show clearly the necessity of considering in the design of arch dams not only the direct compression stresses resulting from the water pressure (ordinary cylinder theory), but also those stresses which are due to deformations and corresponding deflections of the arches and cantilevers.

The deflection of an arch dam may be due to any reason, such as direct water pressure, temperature, shrinkage, swelling, lateral deformation (Poisson's ratio), etc., or a combination of any or all of these. The fact that such deflections have been measured enables the determination of the resulting arch deflection stresses, by the aid of the formulas given, with a fair degree of accuracy.

It has been claimed recently that whenever it takes a comparatively long time, say, 14 days or more, to develop the full load in a concrete dam, the stresses, under such circumstances, would be greatly reduced due to the

so-called "time factor". It is a fact borne out by tests that "green" concrete, and, to a certain degree, also, older concrete, deforms under high compression stresses somewhat more than the assumed laws of true elasticity would permit. Such test results, however, which were made under conditions absolutely strange to what ordinarily is the rule for arch dams, can hardly be applied directly to dam construction, and, as far as the writer knows, they also are not considered in any other practical design of concrete structures. As a matter of fact, experience has shown that in dam construction, in an unmistakable manner, the "time factor" is of negligible influence as long as the unit stresses are low, inasmuch as it is not even able to prevent the very slowly working shrinkage deformations from cracking the concrete. It is the writer's opinion that for these reasons the "time factor" should not be considered in the design of arch dams, as long as the stresses are kept below a reasonable limit.

It is evident that the foregoing deflection-stress formulas are of considerable value also for the design of new arch dams. It is possible in many cases to predetermine the probable deflections of a dam designed as a "cantilever arch dam" by the method of combined cantilever and arch action. Consequently, the "arch deflection stresses" in such a structure may also be predetermined with a fair degree of accuracy. On the other hand, those formulas will enable an approximate determination to be made of the stresses in the arches of existing dams if the deflections are measured. This will then permit engineers to judge the factor of safety of such dams, and also to draw conclusions with regard to the stresses that may be expected in future similar structures.

This fact emphasizes most forcefully the great desirability of having deflection measurements made for all arch dams, and of interpreting such deflections mathematically, for instance, in the manner pointed out in the paper. Such investigations and a thorough discussion of the results will then lead to definite and closely defined safe assumptions for the future construction of arch dams, and the result will be not only a great economy in the cost of such structures, but also a uniform and definite factor of safety.

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THE CIRCULAR ARCH UNDER NORMAL LOADS*

By William Cain,† M. Am. Soc. C. E.

Synopsis.

In the theory of curved dams, the deflection of a horizontal circular arch under normal loads plays an important rôle. The main object of the following investigation is to derive nearly exact formulas for this deflection, as well as formulas for moment, thrust, and shear, both for arches "fixed" (encastré) at the ends and for those free to turn or "hinged" at the abutments. Since the only approximation introduced consists in neglecting the influence of shear on the deformation, the results may be characterized as nearly exact.

It is left to the engineer's judgment to choose the results best adapted to the conditions at the abutments as actually constructed. For thin dams, the theory pertaining to "hinged ends" may be nearer the truth; for thick dams, the hypothesis of "fixed ends" may be more nearly realized. In case anchoring with steel bars to the side-hills is resorted to, the conditions there become more definite.

The formulas being necessarily long, a number of tables of numerical coefficients have been prepared to aid in a quick computation of deflections, or for use where deflections are given by observation and stresses are to be ascertained.

In the analysis of arched dams, the dam is supposed to be divided into a series of horizontal arches, each 1 ft. in depth, and free to move over each other,‡ and the solution is based on the principle that the deflection of any arch and of the corresponding vertical cantilever at its crown, must be the same. The part of the normal load borne by each arch in turn is thus ascertained; and it is this load, acting in a radial direction, that figures in all the computations that follow.

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* This paper will not be presented for discussion at any meeting of the Society, but written communications on the subject are invited for subsequent publication in *Proceedings* and with the paper in *Transactions*.

† Chapel Hill, N. C.

‡ On this point, see the writer's discussion, in Pamphlet 20-C-2, Am. Soc. C. E. (December, 1920), of the paper entitled "Gravity and Arch Action in Curved Dams", by Fred A. Noetzli, Assoc. M. Am. Soc. C. E.

Very simple formulas for the deflection of a horizontal arch subjected to normal loads, have been derived by B. A. Smith, M. Am. Soc. C. E.,* and by F. A. Noetzli, Assoc. M. Am. Soc. C. E.†

Using the notation given in connection with Fig. 1, the crown deflection η, can be reduced to the forms:

$$\text{(Smith)}, \quad \eta = \frac{3}{2}\,\frac{p'\,r'^2}{E\,t},$$

$$\text{(Noetzli)}, \quad \eta = 1.56\,\frac{p'\,r\,r'}{E\,t}.$$

These formulas were derived, for arches "hinged at the ends", by considering the deformations due only to the circumferential thrust, the influence of bending moments being neglected. They are consequently only approximate.

In Table 1 are given the numerical coefficients corresponding to the (nearly) exact solutions of Equations (15) and (19). By comparison, it is seen that the coefficient 1.56 of Mr. Noetzli's formula is sufficiently near for thin dams with "hinged ends" for the usual central angles; but for thick dams and small central angles, the approximate solution gives very erroneous results. The differences are still more pronounced when the arch is really "fixed at the ends."

TABLE 1.—CIRCULAR ARCH FIXED AT ENDS, UNDER NORMAL LOADS.

Values of Coefficient c in $\eta = c\left(\frac{p\,r^2}{E\,t}\right) = c\left(\frac{p'\,r\,r'}{E\,t}\right)$.

$2\phi_1$	$\frac{t}{r} = 0.02$	$\frac{t}{r} = 0.06$	$\frac{t}{r} = 0.10$	$\frac{t}{r} = 0.15$	$\frac{t}{r} = 0.20$	$\frac{t}{r} = 0.25$	$\frac{t}{r} = 0.30$
40°	1.708	0.994	0.542	0.287	0.173	0.115	0.081
60°	1.845	1.606	1.277	0.911	0.651	0.477	0.360
90°	1.879	1.828	1.735	1.577	1.400	1.223	1.060
120°	1.894	1.878	1.848	1.794	1.723	1.640	1.549
180°	1.918	1.916	1.911	1.903	1.891	1.877	1.859

CIRCULAR ARCH HINGED AT ENDS, UNDER NORMAL LOADS.

Values of Coefficient c in $\eta = c\left(\frac{p\,r^2}{E\,t}\right) = c\left(\frac{p'\,r\,r'}{E\,t}\right)$.

$2\phi_1$	$\frac{t}{r} = 0.02$	$\frac{t}{r} = 0.06$	$\frac{t}{r} = 0.10$	$\frac{t}{r} = 0.15$	$\frac{t}{r} = 0.20$	$\frac{t}{r} = 0.25$	$\frac{t}{r} = 0.30$
40°	1.541	1.363	1.108	0.811	0.590	0.437	0.332
60°	1.571	1.518	1.453	1.331	1.190	1.048	0.914
90°	1.578	1.568	1.556	1.528	1.490	1.444	1.392
120°	1.593	1.590	1.587	1.578	1.566	1.551	1.533
180°	1.637	1.636	1.635	1.634	1.632	1.630	1.627

For the latter case, the late R. Shirreffs, M. Am. Soc. C. E.‡ derived an approximate formula for deflection which is as complicated as the (nearly)

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

† Pamphlet 20-C, Am. Soc. C. E. (August, 1920).

‡ *Transactions*, Am. Soc. C. E., Vol. LIII (December, 1904), pp. 163–166.

exact formula, Equation (15), and differs from it in its results very materially, so that it should be rejected.

In the ordinary theory, where vertical loads only are considered, the influence of moment and thrust can be separately considered; but, for the uniform normal loads, the investigation that follows will show that this separation is inadmissible, since it gives $M = 0$, throughout. Mr. Shirreffs, in his analysis, attempts this separation and likewise ignores the influence of P on bending, and the compression of the arch due to H', which is really the equivalent of $(p\,r - P_0)$, as given later. These considerations will sufficiently account for the difference in the results mentioned.

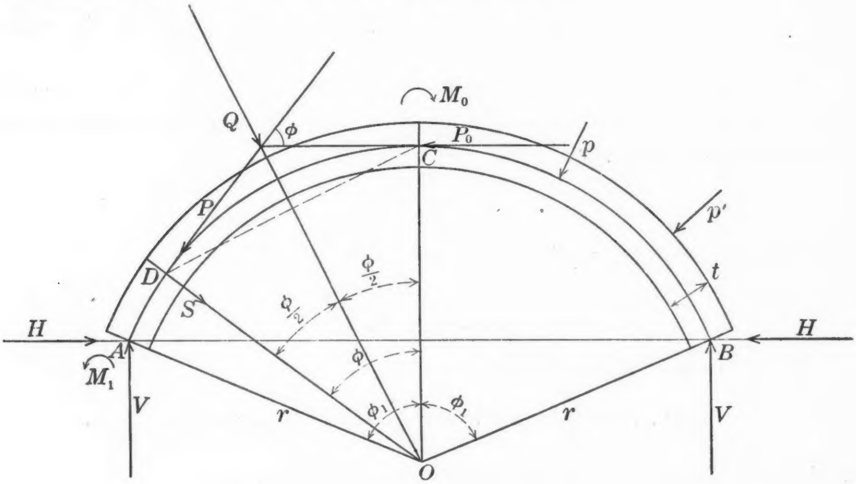

FIG. 1.

CIRCULAR ARCH OF UNIFORM RADIAL THICKNESS, FIXED AT THE ENDS AND SUBJECTED TO A UNIFORM NORMAL, RADIAL PRESSURE.

Fig. 1 is supposed to represent a horizontal circular arch, 1 ft. thick perpendicular to the plane of the paper.

Let, t = uniform radial thickness of arch, in feet;

r = radius of center line of arch, in feet;

r' = radius of extrados, in feet;

p' = normal, radial pressure, in pounds per square foot, on extrados;

p = normal pressure, in pounds per square foot, on center line $= \dfrac{p'\,r'}{r}$;

ϕ = angle with radius of crown for any point, D;

ϕ_1 = half central angle, $A\,O\,B$;

s = length of arc, $C\,D = r\,\phi \;\therefore\; d\,s = r\,d\,\phi$;

E = modulus of elasticity of arch, in pounds per square foot;

M_0 = moment at crown, taken positive clockwise;

P_0 = thrust at crown;

M, P, and S are, respectively, the moment, tangential component of the thrust, and shear at D (r, ϕ).

From symmetry, the thrust, P_0, at the crown, is normal to the radial section there and the shear is zero. At the left abutment the components of the reaction parallel and perpendicular to $O\,C$ are V and H, and the reaction moment is M_1. These forces V, H, M_1, with P_0, M_0, and the load on the semi-arch, hold the latter in equilibrium, so that it can be treated as a free body.

By Merriman's "Hydraulics",* the component, Q, of the loads on the arch segment, $C\,D$, acting perpendicular to the plane, $C\,D = p\,.\,C\,D = p\,.\,2r \sin\frac{\phi}{2}$. Similarly,

$$V = p\,r \sin \phi \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (1)$$

and putting the horizontal components of the forces acting on the semi-arch $= 0$,

$$H = P_0 - p\,r\,(1 - \cos\phi_1) = p\,r\cos\phi_1 - (p\,r - P_0) \ldots\ldots (2)$$

If $I =$ moment of inertia of a radial section at D about an axis perpendicular to the plane of the paper

$$I = \frac{1}{12}\,t^3\,;$$

and the corresponding radius of gyration, k, is given by,

$$k^2 = \frac{I}{\text{area of section}} = \frac{1}{12}\,\frac{t^3}{t \times 1} = \frac{1}{12}\,t^2.$$

The moment of Q about D is

$$Q\,.\,r\sin\frac{\phi}{2} = 2\,p\,r^2\sin^2\frac{\phi}{2} = p\,r^2\,(1 - \cos\phi),$$

and the component of Q in the direction of P is

$$Q\sin\frac{\phi}{2} = p\,r.\,2\sin^2\frac{\phi}{2} = p\,r\,(1 - \cos\phi).$$

On taking moments of the forces acting from C to D about D,

$$M = M_0 + (p\,r - P_0)\,r\,(1 - \cos\phi) \ldots\ldots\ldots\ldots\ldots (3)$$

Also, since P equals the sum of the components of P_0 and Q parallel to P,

$$P = P_0\cos\phi + p\,r\,(1 - \cos\phi) = p\,r - (p\,r - P_0)\cos\phi \ldots\ldots (4)$$

Taking partial derivations of Equations (3) and (4),

$$\frac{\delta\,M}{\delta\,M_0} = 1,\ \frac{\delta\,M}{\delta\,P_0} = -\,r\,(1 - \cos\phi);\ \frac{\delta\,P}{\delta\,M_0} = 0,\ \frac{\delta\,P}{\delta\,P_0} = \cos\phi.$$

The internal elastic work, L, of the semi-arch, neglecting that due to shear as inappreciable, is

$$L = \frac{r}{2}\int_0^{\phi_1}\frac{M^2\,d\,\phi}{E\,I} + \frac{r}{2}\int_0^{\phi_1}\frac{P^2\,d\,\phi}{E\,t} \ldots\ldots\ldots\ldots (5)$$

By the theorem of Castigliano,† the values of the unknowns, M_0, P_0, can be found by putting $\frac{\delta\,L}{\delta\,M_0} = 0$, $\frac{\delta\,L}{\delta\,P_0} = 0$ and solving.

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* Eighth Edition, p. 35.

† "Systèmes Elastiques", p. 265.

From
$$\frac{\delta L}{\delta M_0} = 0,$$
$$\frac{r}{E I} \int_0^{\phi_1} M \frac{\delta M}{\delta M_0} d \phi = 0,$$
therefore,
$$\int_0^{\phi_1} M \, d \, \phi = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$
By aid of Equation (3), this reduces to
$$M_0 \, \phi_1 + (p \, r - P_0) \, r \, (\phi_1 - \sin \phi_1) = 0 \ldots\ldots\ldots\ldots\ldots (7)$$
From
$$\frac{\delta L}{\delta P_0} = 0,$$
$$\frac{1}{I} \int_0^{\phi_1} M \frac{\delta M}{\delta P_0} d \phi + \frac{1}{t} \int_0^{\phi_1} P \frac{\delta P}{\delta P_0} d \phi = 0.$$
On reducing, utilizing Equation (6), and putting $\frac{I}{t} = k^2$,
$$r \int_0^{\phi_1} M \cos \phi \, d \, \phi + k^2 \int_0^{\phi_1} P \cos \phi \, d \, \phi = 0 \ldots\ldots\ldots\ldots (8)$$
On substituting the values of M and P, given by Equations (3) and (4), and integrating, recalling that
$$\int_0^{\phi_1} \cos^2 \phi \, d \, \phi = \frac{\phi_1}{2} + \frac{1}{4} \sin 2 \, \phi_1,$$
$$r \left[M_0 \sin \phi_1 + (p \, r - P_0) \, r \left(\sin \phi_1 - \frac{\phi_1}{2} - \frac{1}{4} \sin 2 \, \phi_1 \right) \right]$$
$$+ k^2 \left[P_0 \left(\frac{\phi_1}{2} + \frac{1}{4} \sin 2 \, \phi_1 \right) + p \, r \left(\sin \phi_1 - \frac{\phi_1}{2} - \frac{1}{4} \sin 2 \, \phi_1 \right) \right] = 0 \ldots (9)$$
To eliminate M_0, multiply Equation (7) by $r \sin \phi$, and Equation (9) by ϕ_1, subtract the last equation from the first, multiply by $2r^2$, and solve for $(pr - P_0)$,
$$(p \, r - P_0) = \frac{p \, r}{D} \, 2 \, \phi_1 \sin \phi_1 \frac{k^2}{r^2} \ldots\ldots\ldots\ldots\ldots (10)$$
where
$$D = \left(1 + \frac{k^2}{r^2} \right) \phi_1 \left(\phi_1 + \frac{1}{2} \sin 2 \, \phi_1 \right) - 2 \sin^2 \phi_1 \ldots\ldots (11)$$
From Equation (10), P_0 can be at once derived. From Equation (7),
$$M_0 = - (p \, r - P_0) \, r \left(1 - \frac{\sin \phi_1}{\phi_1} \right) \ldots\ldots\ldots\ldots\ldots (12)$$
which gives M_0 when $(pr - P_0)$ has been computed from Equation (10). Similarly, from Equations (1) and (2), the components of the reaction at the abutment are ascertained.

If Equation (12) is substituted in Equation (3), we find,
$$M = r \, (p \, r - P_0) \left(\frac{\sin \phi_1}{\phi_1} - \cos \phi \right) \ldots\ldots\ldots\ldots\ldots (13)$$

giving the moment at any point, D (r, ϕ). The value of P there is given by Equation (4).

The shear, S, on the radial section at D will be taken as positive when directed toward the center. The load on the segment, CD, of the arch, due to the normal pressures, p, contributes to this shear a component parallel to $D\,O = p\,r \sin \phi$.* The component of P_0 parallel to $D\,O = -P_0 \sin \phi$; whence

$$S = (p\,r - P_0) \sin \phi \qquad (14)$$

It will be observed from Equations (4), (12), (13), and (14), how persistently the factor $(p\,r - P_0)$ appears; hence, in a numerical example, it is advisable to begin by computing $(p\,r - P_0)$ from Equation (10); whence P_0 can be found, and from Equations (12), (13), (4), and (14), the values of M_0, M, P, and S can then be evaluated. Table 2 (given under "Temperature Stresses"), will prove an aid in the computation of $(p\,r - P_0)$.

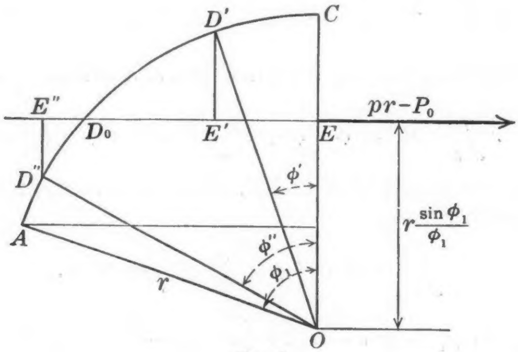

FIG. 2.

The expression for M can be put in a simpler form. In Fig. 2, lay off $(p\,r - P_0)$ as a force acting to the right, perpendicular to the crown radius, $O\,C$ and at a distance $O\,E$ from O equal to $r\,\dfrac{\sin \phi_1}{\phi_1}$, the length (as shown in Mechanics) of the ordinate to the center of gravity of the arc, $A\,C$. Then, from Equation (13), we have

$$M = (p\,r - P_0)\left(r\,\frac{\sin \phi_1}{\phi_1} - r \cos \phi\right) = (p\,r - P_0)\,(O\,E - r \cos \phi)\,;$$

therefore, for $\phi = \phi'$, or point D',

$$M = -\,(p\,r - P_0)\,E'\,D'\,;$$

whereas for D'' $(\phi = \phi'')$,

$$M = +\,(p\,r - P_0)\,D''\,E''.$$

At D_0, where the force line $(pr - P_0)$ meets the arc, $A\,C$, $M = 0$; for points between D_0 and C, M is negative, and decreases numerically in going from C to D_0; for points between D_0 and A, M is positive and increases from zero at D_0 to a maximum at A. From a large scale drawing, the values of $E'\,D'$ and $E''\,D''$ can be scaled off and the values of M quickly computed.

* On the principle (Merriman's "Hydraulics," 8th ed., p. 35) that the component of this load on CD in any direction, equals p times the projection of CD on the perpendicular to this direction, or $p \times r \sin \phi$.

By the theory of the arch, the radial distance from the center of any section, as D', to the center of pressure on that section is equal to $\frac{M}{P}$; hence, since clockwise moments were taken as positive, the line of the centers of pressure meets the crown section on the extrados side of the center line of the arch; it then gradually approaches the center line in going from C to D_0, Fig. 2; at D_0 it crosses the center line, and from D_0 to A_1 it recedes from it increasingly, lying now on the intrados side, and attains its maximum departure from the center line at A. It is true, from Equation (4), that P increases gradually, in going from the crown to the abutment, but not sufficiently so to invalidate the conclusion, as the following numerical illustrations will show:

1.—Let $t = 4$ ft., $r = 135$ ft., and $\phi_1 = 60°$; then, at the crown, $P_0 = 0.99735\, p\, r$, and $M_0 = -0.061965\, p\, r$; whence, $\frac{M_0}{P_0} = -0.0621$ ft., or the center of pressure at the crown is about 0.06 ft. from the center.

2.—Let $t = 40$ ft., $r = 135$ ft., $\phi_1 = 60°$; then $P_0 = 0.024818\, p\, r$, and $M_0 = -2.657282\, p\, r$; whence, $\frac{M_0}{P_0} = -107.07$ ft., or the center of pressure is 107 ft. from the center of the crown section on the extrados side.

In the first example, the thrust, P_0, is nearly $p\, r$, as given by the cylinder formula; whereas, in the second example, P_0 is only about $2\frac{1}{2}\%$ of $p\, r$. It is evident from this how erroneous it would be to assume the cylinder formula for thick arches. In this second example, the arch action is very small; so that the arch acts nearly as a beam fixed at the ends.

The shear, S, as given by Equation (14), is always positive, or directed toward the center. It is zero at the crown ($\phi = 0$), and attains a maximum at the abutment ($\phi = \phi_1$). Consequently, the thrust, $\sqrt{P^2 + S^2}$, on any radial section is not normal to that section, save at the crown, and its component, S, is directed toward the center. The unit stress at extrados or intrados is given by the usual formulas and diagrams, according as the arch is reinforced or non-reinforced.

For brevity, let the symbol $\doteq$ denote "approaches indefinitely" (as a limit); then from Equations (10) and (12), it is seen that as $t \doteq 0$ (and therefore $k \doteq 0$), $p\, r - P_0 \doteq 0$, or $P_0 \doteq p\, r$ and $M_0 \doteq 0$. Also, from Equations (3) and (4), as $t \doteq 0$, $M \doteq 0$ and $P \doteq p\, r$, the "cylinder formula."

The elastic work due to the axial components, P, is given by the term in P in Equation (5), and its influence in the term involving k^2 in Equation (8) and the following equations; hence, at first glance, it would seem that the effect due to M alone could be found by putting $k = 0$ throughout; but, as just seen, this gives $P = p\, r$, $M = 0$ throughout, which is absurd. Thus, the effect of M and P cannot be separated, as is done in the ordinary theory, where vertical loads alone are considered. It appears then, that only a theory like the foregoing, which includes the influence of both M and P throughout, can be expected to effect a solution; and it is on that account, mainly, that Mr. Shirreffs' ingenious solution already referred to fails to give reliable results.

RADIAL DEFLECTION AT THE CROWN, FIXED ENDS.

In the case of the arch, Fig. 1, "fixed at the ends", let a small additional load, w, acting in the direction, $C\,O$, be supposed to be applied to the left half arch at the crown. Designating by M' and P', the new moment and thrust at D, we have only to add to M and P, as given by Equations (3) and (4), the proper terms in w. Therefore,

$$M' = M + w\,r \sin \phi \;\therefore\; \frac{\delta\, M'}{\delta\, w} = r \sin \phi,$$

$$P' = P + w \sin \phi \;\therefore\; \frac{\delta\, P'}{\delta\, w} = \sin \phi$$

As before, the left half arch is treated as a cantilever, and it is supposed to be in equilibrium under the action of the load, w, the couple the moment of which is M'_0, and the thrust P'_0—all acting at the crown—the loads, p, and the consequent reactions at the left abutment.

The elastic internal work of deformation of the cantilever is,

$$L' = \frac{r}{2} \int_0^{\phi_1} \frac{M'^2}{E\,I}\, d\,\phi + \frac{r}{2} \int_0^{\phi_1} \frac{P'^2}{E\,t}\, d\,\phi$$

Therefore, by Castigliano's theorem,* the deflection η at the crown, in the direction, CO, is to be found by taking the partial derivative of L' with respect to w and then putting $w = 0$ in the expressions for M' and P', thus reducing them to M and P. Therefore,

$$\eta = \frac{r}{E\,I} \int_0^{\phi_1} M \frac{\delta\, M'}{\delta\, w}\, d\,\phi + \frac{r}{E\,t} \int_0^{\phi_1} P \frac{\delta\, P^1}{\delta\, w}\, d\,\phi$$

$$\frac{E\,I}{r}\,\eta = \int_0^{\phi_1} M\,r \sin \phi\, d\,\phi + k^2 \int_0^{\phi_1} P \sin \phi\, d\,\phi$$

On substituting the values of M and P given by Equations (13) and (4), and integrating, we find,

$$\frac{E\,I}{r}\,\eta = r^2\,(p\,r - P_0) \left[\frac{\sin \phi_1}{\phi_1} (1 - \cos \phi_1) - \frac{1}{2} \left(1 + \frac{k^2}{r^2}\right) \sin^2 \phi_1\right]$$
$$+ k^2\,p\,r\,(1 - \cos \phi_1).$$

On substituting the value of $(p\,r - P_0)$ given by Equation (10), and writing

$$\sin^2 \phi_1 = 1 - \cos^2 \phi_1 = (1 + \cos \phi_1)\,(1 - \cos \phi_1),$$

$$\frac{E\,I}{r}\,\eta = p\,r\,k^2\,(1 - \cos \phi_1)\,\frac{1}{D} \left[2 \sin^2 \phi_1 - \left(1 + \frac{k^2}{r^2}\right)\right.$$

$$\left.\phi_1 \sin \phi_1\,(1 + \cos \phi_1) + D\right].$$

On substituting the value of D given by Equation (11), the last bracket reduces to

$$\left(1 + \frac{k^2}{r^2}\right) \phi_1\,(\phi_1 - \sin \phi_1),$$

* "Systèmes Elastiques", p. 27 or p. 265.

so that finally, on solving for η, we find, after putting $2 \sin^2 \phi_1 = 1 - \cos 2 \phi_1$, $\frac{k^2}{I} = \frac{1}{t}$, and dividing numerator and denominator by $\left(1 + \frac{k^2}{r^2}\right) \phi_1$,

$$\eta = \frac{p\, r^2}{E\, t} \frac{(\phi_1 - \sin \phi_1)(1 - \cos \phi_1)}{\left(\phi_1 + \frac{1}{2} \sin 2 \phi_1\right) - \dfrac{1 - \cos 2 \phi_1}{\phi_1 \left(1 + \frac{k^2}{r^2}\right)}} \quad \ldots\ldots\ldots\ldots (15)$$

The coefficients, c, of $\frac{p\, r^2}{E\, t} \left(= \frac{p'\, r\, r'}{E\, t}\right)$ have been given in Table 1 for various values of $\frac{t}{r}$ and $2 \phi_1$.

In applying this formula, if t and r are in feet and p in pounds per square foot, then E must be expressed in pounds per square foot (not inch). Also, ϕ_1 must be expressed in radians, corresponding to the angle as given in degrees, for which the trigonometric functions are found. Since, generally, $2 \phi_1 > 90°$, use, $\sin 2 \phi_1 = \sin (180° - 2 \phi_1)$, $\cos 2 \phi_1 = - \cos (180° - 2 \phi_1)$. In numerical computations, such a logarithmic table must be used as to give, at least, three significant figures in the denominator. A 5-place table will usually suffice for $2 \phi_1 > 90°$; but for smaller central angles, a 7-place table may be necessary. In fact, for $2 \phi_1 = 20°$, an 8-place table was desirable.*

Since $\frac{k^2}{r^2} = \frac{1}{12} \left(\frac{t}{r}\right)^2$, the values of c in Table 1 remain the same, whatever the values of t and r, provided the ratio, $\frac{t}{r}$, remains unaltered. The factor $\left(1 + \frac{k^2}{r^2}\right)$ in Equation (15) can be written 1 for small values of $\frac{t}{r}$ when $2 \phi_1 > 120°$, but this approximation would lead to large errors for small central angles. Thus, for $t = 4$, $r = 135$, $2 \phi_1 = 20°$, the true value of c is 0.414; whereas, if $\left(1 + \frac{k^2}{r^2}\right)$ is replaced by 1 in Equation (15), the value of c resulting is 1.884. For large values of t $\left(\text{or of } \frac{t}{r}\right)$, the term in k must be retained throughout.

Circular Arch Hinged at Ends.

As before, the arch will be assumed to have a constant thickness, t, and to be subjected to normal loads, p' pounds per square foot on the extrados, or $p = \frac{p'\, r'}{r}$ pounds per square foot along the center line of the arch. The center of the sections at the abutments will be regarded as fixed, but the sections there will be free to turn, or technically, the arch will be "hinged at the ends." In addition to the previous notation, let (x, y) denote the co-ordinates of D

* An admirably arranged 8-place table is the one by Bauschinger and Peters. It is as convenient to use as a 7-place table.

with respect to A for an origin, x being perpendicular and y parallel to $O\ C$, the crown radius.

The Equations (1) to (5) pertaining to Fig. 1 equally apply to the case of hinged ends, Fig. 3.

Since $M = 0$ at the hinge, $\phi = \phi_1$. Therefore, from Equation (3),

$$M_0 = -(p\ r - P_0)\ r\ (1 - \cos \phi_1)$$

which reduces Equation (3) to

$$M = -(p\ r - P_0)\ r\ (\cos \phi - \cos \phi_1) = -(p\ r - P_0)\ y \ldots\ldots (16)$$

The horizontal deflection of the crown is zero; whence $\frac{\delta\ L}{\delta\ P_0} = 0$, which gives the equation just below Equation (7), or,

$$\int_0^{\phi_1} M \frac{\delta\ M}{\delta\ P_0}\ d\ \phi + k^2 \int_0^{\phi_1} P \frac{\delta\ P}{\delta\ P_0}\ d\ \phi = 0 \ldots\ldots\ldots\ldots (17)$$

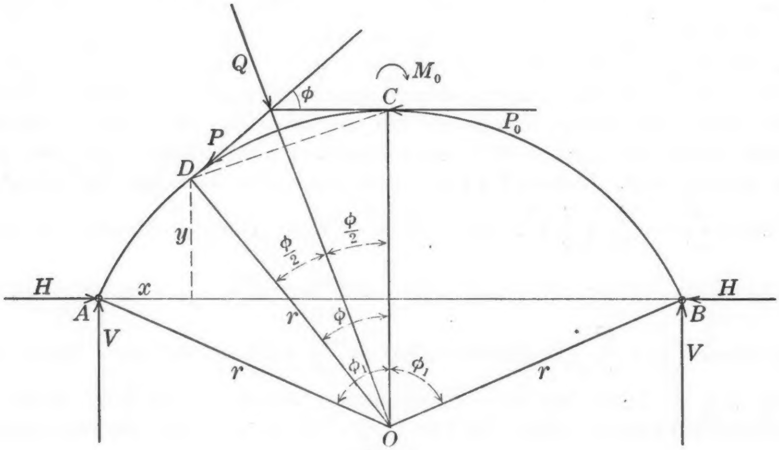

FIG. 3.

Substituting the values of the partial derivatives (given just below Equation (4)), integrating, and solving for $(p\ r - P_0)$, we find

$$p\ r - P_0 = \frac{2 \frac{k^2}{r^2} (p\ r) \sin \phi_1}{\phi_1\ (2 + \cos 2\ \phi_1) - \frac{3}{2} \sin 2\ \phi_1 + \frac{k^2}{r^2}\left(\phi_1 + \frac{1}{2} \sin 2\ \phi_1\right)} \ldots (18)$$

from which P_0 can be computed.

After $(p\ r - P_0)$ has been computed from Equation (18), the values of H, M, and P can be at once written down from Equations (2), (3), and (4).

Since $(p\ r - P_0)$ is always positive, by Equation (16), M is always negative, so that,

$$\frac{M}{P} = -\frac{(p\ r - P_0)\ y}{p\ r - (p\ r - P_0) \cos \phi},$$

is negative, or the line of the centers of pressure is everywhere on the extrados side of the center line of the arch ring except at the abutment,

where it passes through the center of the section. This line increases its distance from the arch axis in going from the abutment to the crown. This arch, with normal loading, is exceptional in that the line of the centers of pressure does not cross the arch axis. The result, however, is perfectly consistent with Equation (17), since the first integral in this equation is negative and the second positive, so that the algebraic sum can be zero.

It is very instructive to compare certain results pertaining, respectively, to thin and thick arches. Thus, let $r = 135$ ft., $2\,\phi_1 = 120°$ for either arch. Then, for $t = 4$ ft., previous formulas give $P_0 = 0.999534\ p\,r$; $M_0 = -\ 0.03145\ p\,r$.

Therefore,

$$\frac{M_0}{P_0} = -\ 0.031 \text{ ft.};$$

whereas, for $t = 40$ ft., we find $P_0 = 0.2\ (p\,r)$; $M_0 = -\ 54.0\ (p\,r)$.

Therefore,

$$\frac{M_0}{P_0} = -\ 270 \text{ ft.}$$

Thus, for $t = 4$ ft., P_0 is nearly equal to $p\ r$, and the center of pressure on the crown section is only 0.031 ft. from its center; whereas, for $t = 40$ ft., P_0 is only 20% of $p\ r$ and the center of pressure on the crown section extended is 270 ft. from the crown, on the extrados side.

It is plain from this that the use of the cylinder formula, $P = p\,r$, for thick arches, will lead to very erroneous results, not only for thrusts and moments, but also for deflections.

Radial Deflection at the Crown, Hinged Ends.

Taking the hinged point, A, Fig. 3, as fixed, regard the hinged end, B, as free to slide on a horizontal plane. Then, if a small radial load, w, is applied at the crown, the reactions due to it, at A and B, will be parallel to $O\ C$, and each will equal $\frac{w}{2}$. The horizontal thrust, H, acting from B toward A, can be supposed to be just sufficient to bring B back to its first position when $w = 0$. The arch is now in equilibrium under the normal loads, p, the reactions, V and H, at the hinged ends, the force, w, at C and the reactions, $\frac{w}{2}$, at A and B. Let M' and P' be the moment and tangential thrust at D corresponding.

The moment, M', at D of all forces and reactions to its right will equal the resisting moment of all forces and reactions on $A\ D$ in magnitude and sign provided the moments of right-handed couples of forces on $A\ D$ are taken as negative, since such moments on $D\ C$ were given the positive sign. Thus, the moment of $\frac{1}{2}\ w$ at A, about D, must be given the minus sign, so that $M' = M - \frac{1}{2}\ w\ x$; where M has the value given by Equation (16).*

* The formula for M was likewise directly proved by considering the forces acting on AD.

Similarly, the tangential component at D acting downward must equal the resisting component acting upward, therefore:

$$M' = M - \frac{1}{2}\, w\, x, \text{ and } \frac{\delta\, M'}{\delta\, w} = -\frac{1}{2}\, x = -\frac{1}{2}\, r\,(\sin \phi_1 - \sin \phi).$$

$$P' = P + \frac{1}{2}\, w \sin \phi, \; \frac{\delta\, P'}{\delta\, w} = \frac{1}{2} \sin \phi.$$

If U is the elastic work for the whole arch, then, neglecting the work of shear,

$$U = 2\left[\frac{1}{2}\int_0^{\phi_1} \frac{M'^2}{E\,I}\, r\, d\,\phi + \frac{1}{2}\int_0^{\phi_1} \frac{P'^2}{E\,t}\, r\, d\,\phi\right];$$

whence the radial displacement, η, at the crown, as $w = 0$, is,

$$\eta = \frac{\delta\, U}{\delta\, w} = 2\int_0^{\phi_1} \frac{M}{E\,I}\,\frac{\delta\, M'}{\delta\, w}\, r\, d\,\phi + 2\int_0^{\phi_1} \frac{P}{E\,t}\,\frac{\delta\, P'}{\delta\, w}\, r\, d\,\phi.$$

On substituting the values of M and P given by Equations (16) and (4) and the values just given for the partial derivatives, we derive,

$$\frac{E\,I}{r^3}\,\eta = \int_0^{\phi_1} (P_0 - p\,r)\,(\cos\phi - \cos\phi_1)\,(\sin\phi - \sin\phi_1)\, d\,\phi$$

$$+ \frac{k^2}{r^2}\int_0^{\phi_1}\Big[(P_0 - p\,r)\sin\phi\cos\phi + p\,r\sin\phi\Big]\, d\,\phi$$

$$= (P_0 - p\,r)\Big[\cos\phi_1\,(\cos\phi_1 - 1) - \frac{1}{2}\sin^2\phi_1 + \phi_1\sin\phi_1\cos\phi_1$$

$$+ \frac{k^2}{r^2}\,\frac{\sin^2\phi_1}{2}\Big] - \frac{k^2}{r^2}\, p\, r\,(\cos\phi_1 - 1).$$

The value of $(P_0 - p\,r)$, as given by Equation (18), is now substituted; after which considerable trigonometric and algebraic reduction* finally leads to the following formula for deflection:

$$\eta = p\, r^2\,\frac{1 - \cos\phi_1}{E\,t\,B}\left[\sin\phi_1 + \phi_1\,(1 - 2\cos\phi_1) + \frac{k^2}{r^2}\,(\phi_1 - \sin\phi_1)\right]. \quad (19)$$

where

$$B = \phi_1\,(2 + \cos 2\,\phi_1) - \frac{3}{2}\sin 2\,\phi_1 + \frac{k^2}{r^2}\left(\phi_1 + \frac{1}{2}\sin 2\,\phi_1\right).$$

Writing this in the form, $\eta = c\,\frac{p\, r^2}{E\,t} = c\,\frac{p'\, r\, r'}{E\,t}$, the coefficients, c, can be computed for varying values of $2\,\phi_1$ (the central angle) and $\frac{t}{r}$. The results are given in Table 1 and have previously been discussed in connection with those pertaining to certain approximate solutions. The general remarks under Equation (15) apply equally here as to ensuring accuracy in the computations.

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* The following formulas were used; $\sin 2\,\phi_1 = 2\sin\phi_1\cos\phi_1$, $2\cos^2\phi_1 = 1 + \cos 2\,\phi_1$, $\sin^2\phi_1 = (1 + \cos\phi_1)\,(1 - \cos\phi_1)$. The factor $(1 - \cos\phi_1)$ will be found to be common to the terms in the numerator and can be taken out.

In the theory of curved dams, as given by B. A. Smith,* M. Am. Soc. C. E., the formula for arch deflection involves a constant coefficient, c, so that an average value for the horizontal arches of varying thickness, for the entire height of the dam, will have to be used. This can be taken with sufficient accuracy, for thin dams, from Table 1. For very high dams, in which the thickness near the base is considerable, the coefficients may vary too much to effect a practical solution by use of an average coefficient. For thin dams, the solution is very satisfactory, and it can be effected, not only where the base of the dam is fixed, but likewise where it is simply supported on the foundation. Mr. Noetzli's tentative method† applies only to the case where the dam is fixed at the base. For this case, an average value of c can be used for thin dams; but for very high dams, the values of c corresponding to the varying values of the thickness, t, should be used. To save labor, a graph of the values of c for different heights, should be made. The solution can then be effected by proceeding along the lines indicated by the writer in his discussion‡ of Mr. Noetzli's paper, either for water loads or for temperature changes; the final test being that the deflections of the supposed horizontal arches and the vertical cantilever, at the crown of the arches, shall be the same at the same depth. The solution gives the values of p for any depth, from which moments, thrusts, shears, and stresses at intrados and extrados can be computed.

Temperature Stresses.

Arch Fixed at the Ends.—If $e =$ expansion per foot for a rise of temperature of 1° Fahr.; then, for t_0 degrees rise for the circular arch above an assumed mean, the thrust, H, acting perpendicular to the crown radial section, is given by the formula,§

$$H = \frac{2 \sin \phi_1}{D_0} \cdot \frac{E\, I\, e\, t_0}{r^2}$$

where

$$D_0 = \left(\phi_1 + \frac{1}{2} \sin 2\, \phi_1\right) \left(1 + \frac{k^2}{r^2}\right) - \frac{1 - \cos 2\, \phi_1}{\phi_1}.$$

Note that D of Equation (11) equals $\phi_1\, D_0$.

The numerical values of $\frac{2 \sin \phi_1}{D_0}$, for various values of $2\, \phi_1$ and $\frac{t}{r}$, are given in Table 2.

The formula for the moment, M, at any point $(r\, \phi)$ of the axis, is

$$M = H\, r \left(\cos \phi - \frac{\sin \phi_1}{\phi_1}\right).$$

It follows, if we conceive H to act to the left at a distance $r \frac{\sin \phi_1}{\phi_1}$, from the center, Fig. 4, that the moment M at a point $(r\, \phi)$, is given by H times

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–1920), p. 2027.

† Pamphlet 20-C, Am. Soc. C. E. (August, 1920).

‡ Pamphlet 20-C-2, Am. Soc. C. E. (December, 1920).

§ Professor Church, in "Mechanics of Internal Work", p. 118, derives the same formula, using H and the moment at an abutment as the unknowns. The writer used H and the moment at the crown as the unknowns, giving a shorter solution.

the perpendicular distance from $(r\ \phi)$ to this line of direction of H, the moments being positive (clockwise), for points between this line and the crown, and negative for the remaining points. This line of direction of H passes through the center of gravity of the arc, $A\ C$.

TABLE 2.

$2\,\phi_1$	$\frac{t}{r} = 0.02$	$\frac{t}{r} = 0.06$	$\frac{t}{r} = 0.10$	$\frac{t}{r} = 0.15$	$\frac{t}{r} = 0.20$	$\frac{t}{r} = 0.25$	$\frac{t}{r} = 0.30$
40°	2 750.4	1 600.0	872.0	461.1	277.9	184.0	130.2
60°	583.7	507.9	403.7	288.4	205.3	150.1	113.2
90°	115.9	112.7	107.0	97.1	86.1	75.0	64.9
120°	36.2	35.9	35.3	34.2	32.8	31.2	29.4
180°	6.7	6.7	6.7	6.6	6.6	6.5	6.5

A remarkable analogy may now be pointed out in the case of water load or normal forces only. Thus, from Equation (10),

$$(p\ r - P_0) = \frac{2 \sin \phi_1}{D_0} \left(p\ r\ \frac{k^2}{r^2} \right),$$

and it is seen that the values of $\frac{2 \sin \phi_1}{D_0}$ are precisely those given in Table 2; so that the values of $(p\ r - P_0)$ and, thus of P_0, can be at once written down for the values of $2\ \phi_1$ and $\frac{t}{r}$ given.

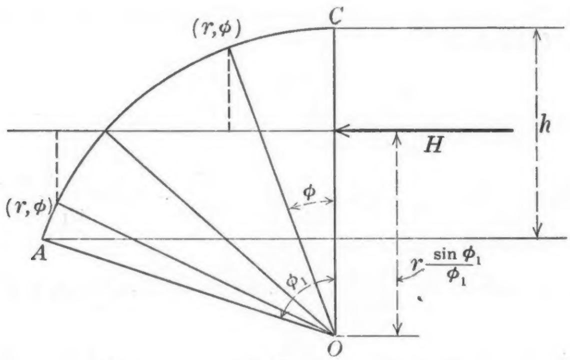

FIG. 4.

Further, if $(p\,r - P_0)$ is taken as a force, with the line of action of H in Fig. 4 only acting to the right, then, since by Equation (13),

$$M = -(p\ r - P_0)\ r \left(\cos\ \phi - \frac{\sin \phi_1}{\phi_1} \right),$$

the moment at any point $(r\ \phi)$, is equal to $(p\ r - P_0)$ multiplied by the perpendicular distance from $(r\ \phi)$ to the supposed line of action of $(p\ r - P_0)$. This striking property of the circular arch subjected to normal loads, has already been brought out in connection with Fig. 2.

If we let $h = r\,(1 - \cos\phi_1)$ equal the rise of the center line of the arch, then the formula for the crown deflection due to a temperature rise of t_0 degrees Fahr. is,

$$\eta_1 = \frac{h\,e\,t_0 \sin\phi_1}{D_0}\left[(1+\cos\phi_1)\left(1+\frac{k^2}{r^2}\right) - 2\,\frac{\sin\phi_1}{\phi_1}\right]$$

Arch Hinged at Ends.—For a rise of temperature of t_0 degrees Fahr., the thrust, H, acting inward at an abutment along the span line is,*

$$H = \frac{E\,I}{r^2\,B}\,2\,e\,t_0 \sin\phi_1$$

where,

$$B = \phi_1\,(2 + \cos 2\,\phi_1) - \frac{3}{2}\sin 2\,\phi_1 + \frac{k^2}{r^2}\left(\phi_1 + \frac{1}{2}\sin 2\,\phi_1\right)$$

The moment at any point (r, ϕ) of the axis of the arch is,

$$M = H\,r\,(\cos\phi - \cos\phi_1).$$

The radial deflection at the crown is

$$\eta = \frac{h\,e\,t_0}{B}\left[(1+\cos\phi_1)\left(2\,\phi_1\cos\phi_1 + \frac{k^2}{r^2}\sin\phi_1\right) - \sin\phi_1\,(1 + 3\cos\phi_1)\right]$$

The derivation of the formulas for deflection is so similar to that for normal forces that it is omitted.

When the Crown Deflection is Given by Observation.—From formulas given, by expressing H (or the values of $(p\,r - P_0)$ in the case of normal forces) in terms of η $\left(\text{by first writing the ratios } \frac{H}{\eta} \text{ or } \frac{p\,r - P_0}{\eta}\right)$ and substituting in the formula for M, formulas are derived of the form,

$$M = a\left(\frac{E\,t^3}{r\,h}\right)\eta$$

At the crown, $\phi = 0$, $M = M_0$, the values of the coefficients a, when the term in k is ignored, referring to the four cases indicated, are given in Table 3. The results are applicable to the crests of many curved dams,

TABLE 3.

$2\,\phi_1$	Fixed Ends.		Hinged Ends.	
	Water pressure.	Temperature.	Water pressure.	Temperature.
60°	0.159	0.170	0.096	0.105
90°	0.150	0.180	0.090	0.111
120°	0.138	0.188	0.083	0.121

where t is small and k can be ignored in the formulas. When η is given by observation, M_0 can be computed, if the influence of water pressure and temperature change can be separately estimated.

As dams are usually constructed, the ends are neither fixed nor hinged, so that the engineer is forced to use his judgment in selecting a coefficient,

* Professor Church gives one derivation of this formula in his "Mechanics of Internal Work", p. 109.

which should incline toward that for fixed or hinged ends according to the thickness of the arch and the conditions at the abutments.

Generally, the deflection η has been measured for a combined water pressure and change of temperature. If the deflections for each cannot be separately estimated, still some idea of the value of M_0 can be had, for a given $2\,\phi_1$, by taking for a a rough average of the four coefficients, having regard to the conditions at the ends.

To the stresses at the crown corresponding to the value of M_0 must be added the stresses due to P_0 or H. To estimate P_0 (the thrust at the crown due to water pressure), it is necessary to know p, the unit normal pressure carried by the horizontal arch at the crest. No rule can be given for any dam; but, for the Wooling Dam, p is roughly equal to the full water pressure at one-sixth or one-third of the depth of the dam below the crest, according as the dam is fixed or not fixed at the base.* If a complete solution has been made, as in the case of the Wooling Dam, then p is known and P_0 can be computed.

* See Tables 5 and 11 of the writer's discussion of Mr. Noetzli's paper on "Gravity and Arch Action in Curved Dams", Pamphlet 20-C-2, Am. Soc. C. E. (December, 1920). Also, some information can be drawn from Tables 8 and 12, as to estimating H, the thrust due to change of temperature.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

NATIONAL PORT PROBLEMS*

A SYMPOSIUM

By Messrs. Frederick W. Cowie, Lansing H. Beach, Frederic H. Fay, M. A. Long, J. Rowland Bibbins, John Meigs, W. Watters Pagon, Edwin J. Clapp, Carroll R. Thompson, John A. Bensel, William J. Wilgus, B. F. Cresson, Jr., and H. McL. Harding.

With Discussion by Messrs. William H. Adams, Arthur M. Shaw, T. F. Keller, Harwood Frost, L. F. Bellinger, John H. McCallum, Frank W. Hodgdon, M. G. Barnes, Nelson P. Lewis, and T. Howard Barnes.

* Presented at the meetings of September 7th and 8th, 1921.

NATIONAL PHASES OF PORT PROBLEMS

By Frederick W. Cowie,* M. Am. Soc. C. E.

What is a National port? National ports stand for policy, also projects.

In recent years, increasing productiveness, the tremendous increase in foreign commerce, and the increase in population, wealth, and industrial activity, has not been followed, or even equalled, by the commensurate, scientific development of ports.

The transportation problem of a nation is one of the fundamentally vital economic questions of the day. It is not one of limitations as regards the individual or the locality. It gives additional prices to the farmer for his produce; it enables the manufacturer to compete with his foreign rivals; it cheapens the necessities of life; and it gives employment to labor and capital alike. A successful National port, therefore, is widespread in its effect for good. It may and frequently does constitute the deciding factor as regards a competitively successful National route.

A National port is an important link in the transportation system of a nation. A successful National port is a distinct aid in the solution of the transportation problems of a country.

A National port must bear a clear relation to a logical, economical, and widespread necessity. To be truly National, a port, or series of ports, must be required by the nation and directly or indirectly it must yield comprehensive advantages. In consequence, few port locations fulfill the necessary requirements. A port, however, may be National in part for comprehensive collective and distributing functions, and local in part to fulfill city or State requirements.

Although a National port is of general value to the nation, it is obviously of particular value to the city in which it is located and to the hinterland which it serves. The district through which the transportation route runs collects transportation tolls all along the line. To distribute the benefits and to divide the burdens alike has ever been the difficulty.

A State, a city, or even a corporation, may decide, in view of expected financial and commercial results, to develop an advantageous natural harbor, even though no great comprehensive or National economic interests would be served. In no sense could this be considered a National port. On the other hand, it would be logical to infer that a comprehensive unit of a port may be developed to meet National requirements and, in the same harbor, other units to meet local or more city requirements. A combination port developed and administered, alike for National and for restricted requirements, both as to benefits and the assumption of financial burdens, would give ideal results. The one may not be successful without the other. The combination results in excellent working conditions.

An island nation having only short distances to the seacoast may be better served by a number of local ports. Transportation is not a problem in this case. No comprehensive general interests would be served by the development of one or more particular ports. All producers and consumers have the

* Montreal, Que., Canada.

competitive advantages of short transportation distances and cheap freights. On the contrary, a country having an extensive hinterland and a short sea-coast and having rival, foreign, competitive transportation routes, essentially requires a system of National ports. The transportation problem in this case is to find the best National route, develop it, cheapen it, and use it. Transportation tolls are kept in the country and constitute a National advantage even though the foreign route may be cheaper to some extent. Many rich hinterland areas cannot be developed successfully or opened up for settlement and production, without National aid toward cheap transportation. In particular, therefore, a country having immense transportation distances and productive hinterland areas, vitally requires a National transportation policy, with more or less clearly defined National routes, wisely developed National ports, scientifically improved, equipped, and operated in the interests of a nation. Such examples as the United States, Canada, Brazil, Russia, and Germany may well be cited.

In the United States the genius of commerce is one of the important characteristics of the times. It is a function of the nation rather than of the individual. National prosperity is vitally dependent on commerce. Transportation is one of the important phases of commerce. A transportation policy is essential. This policy, as to its National well-being, may be compared with its fiscal or banking policy. Production may be hampered; manufacturers may not be able to compete; the necessities of life may cost too much, if a satisfactory solution of the transportation problem is not achieved.

The so-called "pork barrel policy" is directly and indirectly the antithesis of the National port policy. If certain ports, natural or artificial, can, by scientific development, be made by the nation of value to the people in general, such development will be in line with a National port policy. It does not follow, however, that because of National expenditure on these certain ports, similar or proportionate National expenditure must be made on others which are not so essential in connection with the adopted National routes.

National ports must be connected with trunk lines, not with local lines or feeders. Is it reasonable to expect that a nation can be asked to sanction such a National port policy which, though general in its good results, must be more generous to some ports than to others? History demonstrates an affirmative reply. A necessary project for the country will command universal approval, even at some sacrifice of the few. Admiral William S. Benson, U. S. N., is recorded as saying, after seeing other harbors and ports of the world: "We [the United States] have not really a first-class harbor, *viz.*, scientifically constructed and developed".

The speaker has seen and studied most of the great harbors of the world and, in his opinion, no country, other than the United States, except possibly Canada, has greater necessity for scientifically developed harbors and. in no country of the world, can be found more favorable opportunities. Transportation is fundamentally essential; distances are great; and routes are well defined. A single great city in the United States possesses two of the most magnificent railway passenger terminals in the world. They are so splendid, so successful, so representative of the spirit of the people, that it is

incomprehensible that there is not one or more similarly representative ocean terminals. It may not be a nation's business to build something splendid for one city, or one State, or one steamship line, or one railway system, but a splendid ocean terminal in the greatest port in the world, at the gateway to three-quarters of the passenger commerce of the country, is a sane vision. Moreover, if properly co-ordinated, with a comprehensive National port unit, it would be an extraordinarily attractive and paying proposition.

Canada, with her great central but rich productive areas, her necessarily long east-and-west transportation, and her limited financial resources, is making wonderfully successful progress. There are notable examples of National ports in Canada. There are others partly National and partly local and they are making wonderful strides in helping to solve that most critical Canadian effort, *viz.*, the development of her vast areas in order to enable her people to compete in production and industry with other nations of the world.

Brazil is in the process of development and is successfully exploiting the nebulæ of what in time will become an organization which will result in the successful development of a great country.

The case of Russia is at present desperate. When the regeneration comes, if that vast country is to be successfully developed, a series of National ports on the east, south, and west will be one of her first necessities.

The Port of Hamburg, with its auxiliary, Cuxhaven, is reported to have cost the State of Hamburg little short of $100 000 000, and that the dues collected did not nearly pay the expenses of the port system. It was, however, a National port, and the deficit was cheerfully met from other sources of revenue. A similar courageous and wise policy of port development in America would surely result in widespread and remunerative trade and commerce, both in the United States and in Canada. There is no example in ancient or modern times of such a proportionate trade and commerce as was developed in Germany from 1888 to 1914. A settled purpose made itself manifest; a National transportation policy was adopted; routes were developed and encouraged for the purpose of increasing trade and for conducting it within the limits of the Empire. National privileges were granted to certain ports making them National ports, and the Empire prospered.

A National port, therefore, may be defined as one of the most important factors in the solution of the National transportation problem. This problem does not exist in certain countries. It is of vital economic importance for the prosperity and development of other countries having large productive areas and long transportation distances. What question would require more serious study, greater skill, or be more in the general public interest? The engineer has great responsibilities; he should be well posted when called on for advice.

Accommodation and Facilities Required by a National Port.—In order to measure justly what facilities may be required to take care of National port business, the characteristics of that business may be enumerated:

1. Ocean passenger business.
2. Mails and express.
3. Exports.
4. Imports.

5. The storage and handling of grain.
6. The storage and handling of perishable products.
7. General warehousing.
8. Adjuncts necessary for an ocean port.
9. Facilities and accommodation for collecting, receiving, assembling, re-manufacturing, conditioning, and distributing both from and to all ports of the world and from all points in the country.

Reduced to a single picture, the requirements for a National port may be visualized as follows: One or more comprehensive units capable of taking care of all the varied port business, co-ordinated and concentrated in a central location on one of the main natural routes, where the ship and the railway meet and where commerce may be collected, stored, and handled with economy and dispatch. It would appear, therefore, if this argument is accepted, that much of the port development of the present day is being carried out on lines absolutely contrary to the best requirements of a National transportation policy. Take location: Are the best, most centrally located sites for port development being secured no matter what the cost? Are comprehensive units being designed, so that everything required in connection with the prompt and economical loading and unloading of ships may result? Is concentration being adopted as a necessary principle? What about co-ordination? Who is responsible for the railroads designing ocean terminals?

In North America the super-skill of the designers of everything pertaining to railroads, bridges, the development and use of electricity, the motor industry, bulk-cargo handling, sky-scraper buildings, and tunnels is an acknowledged fact. Is this super-skill in evidence in the ports?

New York which possesses the greatest port in the world and where the lead is taken in almost every phase of originality and skill in design, is, in the opinion of many, setting a very bad example in the development of her port from the standpoint of the inland producer and consumer. The local trade of the great city is such a valuable freight and such a high proportion of the port business, that ships and shipping declare for New York, in spite of the records of economy and dispatch. Competition has not been a factor and has resulted in indifference. Necessity did not require to mother invention.

The port problem in New York, as compared with other successful National ports, such as Hamburg and even Montreal, also appears to be so simple. What is the area of the Port of New York? How many miles of water-front has it? It is reported that in the combined Port of New York and New Jersey, the harbor shore line is about 700 miles in length. The developed water-front is 380 miles long.

The Port of Hamburg covers an area, 4 miles long by 2 miles wide. The harbor of Montreal has a shore-line water-front of 34 miles, while the developed water-front is only about 12 miles in length. In Montreal, centralization is one of the essential features of success. The value of port locations or berths may be judged from wharfage revenues on goods. A pier, dock, or transit shed, and dredging, costs no more in a central location than in an outlying district. The land or shore areas required are tremendously more

costly, but modern facilities may be designed for such sites, which, by concentrating effort, may multiply usage, with good financial results.

In the twenty fully equipped berths in the harbor of Montreal, centrally located along a mile of shore front, more than 75% of the business of the harbor is carried on. The revenue from wharfages on goods alone amounts to between $40 and $50 per lin. ft. of dock front per annum, including ends, bulkheads, etc. A mile distant, where railway and city approaches are not so favorable, but where docks, dredging, wharf spaces, roadways, etc., cost just as much, the wharfages amount only to approximately $10 per lin. ft. per annum. Three miles distant, with excellent railway and city approaches, with distance as the only drawback, the revenue return is about $12 per lin. ft. per annum.

To "centralize" a summing up: The Port of Montreal records a commerce in seven months, of between one-fifth and one-sixth of the commerce of the Port of New York in twelve months, and this without congestion and with a water-front development of one-thirtieth of that of New York. In New York, as in several of the other ports doing National business, there is ample opportunity, there is necessity of centralization, concentration, and comprehensive facilities for economy and dispatch for the Nation's business.

Whose Function is the Development, Construction, and Operation of a National Port?—Assuming that a nation adopts a National transportation policy, including the system of National ports, it is logical that the nation, through its representatives, has the deciding voice in all matters pertaining to policy, finance, and control. There are various principles which may be adopted in the carrying out of this policy. The matter may be placed in the hands of a Government department; it may be placed in the hands of the engineers of the Army; or it may be taken care of by direct legislation.

In order that the policy may be absolutely impartial and in the interests only of the country in general, the consideration of this transportation policy should be placed first, last, and all the time, in the hands of men who have no special interests to serve, and who have no political aspirations. Experiences during the World War are fresh in the memory. Organization after organization under the stress of vital requirements was made subservient to new methods, dependent on direction by men who had been unusually successful in some line of business or commercial activity. This resulted in an unexampled success. Problems previously considered open for individual effort, for gain, were relegated to National conferences of mind and effort. The "dollar-a-year men" were available for the critical requirements of the war. They are ever available for honorable positions at the country's call. Theirs would be the hands into which the transportation problem could safely be confided.

Although men who have already made their mark commercially or financially may be found, who will assume an honorable position and "carry on" for the honor of their country, there is another equally important phase in the success of a National port system. Port administration and port engineering are tremendous factors affecting the success or failure of such plans as would enter into the complete National system. It is probable that in no other

branch of engineering can be found so many costly mistakes. It is probable that in no other branch of engineering have the principles of success and failure in the past been so overlooked. It is a fact that no engineering study, technical or practical, can be found so absorbing, so interesting, and so valuable, when properly developed principles are carefully adhered to. In Europe, where competition in ports is so keen and where ports in general are carried on as public obligations, port engineering ranks among the very highest in the Profession.

Furthermore, the questions of continuity and permanence in connection with the conception and scope of port projects cannot be successfully answered by a system where administrators, engineers, and other technical and practical officers are changed at frequent intervals. The European system by which the best men are obtained, and permanently retained, by which, competitively, they are called on to make good and when successful are free to act independently and according to their best judgment, would appear to be a necessary principle in National port organization.

It would appear, therefore, that the development, construction, and operation of a National port system should be considered first in connection with a National transportation policy. This policy must necessarily be under the direction of the representatives of the people. Impartial direction must be given as to its magnitude, scope, and carrying out. This organization, to be truly National, should be undertaken as a duty, with public honor as the reward. Great public corporations are so directed by men who have successfully developed the various phases of success in commercial life. Successful men, in developing and building up the country's business, may be found to formulate and recommend for legislative approval a transportation policy and to direct its administration.

As a successful port owes much of its success to an advantageous location, it follows that the authorities of that location, therefore, have a vested voice in its development and operation. The hinterland served by the port has at least an equal voice, as providing the commerce for its revenues and a share in its cost, and because of its interest in economy and dispatch. Subject to the administration of the National Board, the various units of a National port system would have full local control and carry out the National units in co-ordination with the local or city port facilities.

From the point of view of the speaker, therefore, after a life-long study of the design and operation of ports, it would appear that a country desiring the adoption of a National port system, should place the development of the policy in the hands of successful men, who for the honor of their country are willing to serve, and their policy and recommendations should be authorized according to an approved programme.

Each National port scheme, adopted by the National commission, should be placed under a local commission having one representative from the hinterland, one from the line of route, and a chairman from the port city. This local commission, subject to the programme of development, expenditure, and administration, should be free and independent and should be remunerated according to its duties and responsibilities. Cheap money would be avail-

able. As far as possible, each unit should operate at cost, but the nation, which would reap the benefits, should be responsible for such interest charges as are not met. It is a recognized principle that in a plan, which is essential to the well-being of the nation, and which cannot be financially developed by a city or a corporation, must be developed, either directly by the nation, or indirectly by some aid which will enable it to stand on its own feet.

Cheaper and more prompt transportation is required by the people in America. If corporations, cities, or the State, do not provide the remedy, it will not be long before the farmers, producers, and industrial interests of the interior, take a hand. The coast cities have had their turn. If the country is not satisfied, the National port policy is open for a trial.

It might reasonably be suggested that it would be in order to outline that which would constitute a central concentrated, co-ordinated and comprehensive unit, which would be for the well-being of a nation.

In the first place, the location must be impartially chosen, and it must be on a National route. It is most desirable that the unit should be in the very center of a harbor, close to business and trade originations. Approaches from ocean routes, connections with all railway lines, and access from city traffic centers should be perfect, or possible of being made so.

To be self-contained the unit should comprehend: A bulkhead passenger landing quay, having berths for at least two of the largest ocean vessels on the route, approachable at all stages of the tide, and connected with a railway passenger-and-mail special siding.

The freight accommodation should be sufficient for at least twenty-five first-class berths, all closely co-ordinated. Each berth should have transit sheds, with accommodations and facilities for entering, unloading, loading, and clearing in five days.

Railway accommodation, and vehicular and lighterage facilities should be ample and sufficient for the aforesaid traffic. Warehouses should be provided for all staple products, such as for the collection, storage, and handling of grain, flour, cotton, sugar, tobacco, etc., all connected for mechanical handling to and from the ships, and with railway cars.

Cold-storage facilities should also be provided so as to develop production in the country, and to cheapen the essentials for the population.

The high priced central sites could be made revenue producing by constructing upward, for offices, manufacturing lofts, sales rooms, exhibitions, etc. The whole unit should be of a permanent character and absolutely fireproof, in order to command low insurance rates. The style and types should be worthy of the nation. If this is what a country wants, it will have it.

TERMINALS

By Lansing H. Beach,* M. Am. Soc. C. E.

It seems proper to explain that this subject was intentionally adopted without qualifications in order to give latitude for the discussion of such phases as might appear most serviceable to the Profession. Since selecting the title the speaker finds that certain special aspects of the terminal problem are also to be discussed. Accordingly, he will confine his remarks largely to the duties respecting terminals as imposed by law on the War Department and on the Corps of Engineers, and in explaining that attitude toward the port and terminal situation in general.

A seaport terminal may be considered to consist of land approaches, whether by rail or highway, of the terminal structures proper, including the wharf, pier, or quay, the transit shed and the necessary supporting warehouses, and the water approach. Speaking for the moment only of the latter, it is the policy of the United States Government to limit itself to work on the outer or channel side of the main harbor line, leaving to the owner of the terminal in question the making of any needed connection between the main channel and the face of the wharf or quay.

As a rule, the Government creates the main channels, usually in the past without any assistance from the locality concerned, and, once these channels have been completed, maintains them, and improves and adds to them from time to time as the demonstrated needs of traffic may demand.

The admirable and comprehensive report of the New York-New Jersey Port and Harbor Development Commission is no doubt familiar to engineers, and its plans and general recommendations are known. Having in mind what is proposed in the way of future developments in the Metropolitan District, it may be interesting to summarize briefly the principal channels already created and maintained by the United States. These include the Ambrose Channel, which is 40 ft. deep and 2 000 ft. wide; the Gedney, or Main Ship Channel, which is 30 ft. deep and 1 000 ft. wide; the Coney Island Channel, 20 ft. deep and 600 ft. wide; the Bay Ridge, Red Hook, and Navy Yard Channels, on the Brooklyn water-front, 40 ft. deep and from 1 000 to 1 200 ft. wide, this channel being now in the process of extension through Hell Gate with the same authorized depth, although, at present, excavation at Hell Gate is being limited to a 35-ft. depth by 500-ft. width. The Shell Reef Channel on the New York side of the East River, 25 ft. deep and of varying width, is intended to afford useful access to that portion of the Manhattan water-front so that it may soon be improved with appropriate terminals. The Hudson River channel, on the west side, is 30 ft. deep at Jersey City, 40 ft. deep at Hoboken, and 26 ft. at Weehawken. Obviously, the projects for these later improvements should eventually be unified, but, on the whole, it may be said that the channels so far described afford ample facilities for approaching the various portions of the water-front to which they relate.

It will be remembered that in the report of the New York-New Jersey Commission recommendation is made for the development of Jamaica

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Bay, largely for industrial purposes. The project for Jamaica Bay, adopted many years ago, is a co-operative one, under which the Government agreed eventually to provide an entrance channel, 30 ft. deep and 1 500 ft. wide, protected by one jetty, or, if necessary, by two jetties, and an inner channel, 30 ft. deep and 1 000 ft. wide, is to be provided by the City of New York, the United States to pay such portion of the cost of dredging the interior channel not in excess of 8 cents per cu. yd. The City further undertook to construct certain bulkheads, make certain slips, and create such fills from the available spoil as might be necessary to reclaim the lowlands bordering the Bay and to make them available for terminal development under a plan that had been proposed on its behalf by a technical commission retained for that purpose. At present, the work that has been done on this project is limited to an entrance channel 18 ft. deep and 500 ft. wide, extending inland 8 800 ft. to Mill Basin and, as yet, the City has not been able to fulfill the terms of the original co-operative agreement. Recently, however, the representatives of the City have been in conference with the agents of the United States with a view to securing early and effective action toward the completion of a considerable portion of the main 30-foot channel previously described. The matter is more fully discussed in Document No. 4 of the Committee on Rivers and Harbors, House of Representatives, 66th Congress, Second Session, in which it is pointed out that further action in regard to Jamaica Bay should be deferred until its development as an element in the progressive improvement of the entire Port of New York has been considered and decided, the underlying principle being that any comprehensive plan for the further development of the Port of New York would certainly indicate the relative importance of its various elements, and that Jamaica Bay should progress in accordance with the orderly programme thus laid down.

As far as such a project as Jamaica Bay is concerned, however, it may be of interest to engineers to know that in recent years it has been the wholesome policy of Congress to require greater local co-operation toward paying the cost of main channels and of such protective works as are necessary to insure the integrity of these main channels whenever the project to be undertaken is of a more or less speculative character, as is true, for example, in the case of Jamaica Bay where no commerce exists and where the character and extent of future traffic are, after all, more or less matters of opinion. It may well be that when in the future the larger project for making available the waters and shores of Jamaica Bay again comes up for consideration, Congress will feel justified in asking the community to bear its fair share of the risk, not only by improving real estate, but also by paying a fair portion of the cost of channels which in the end may possibly prove to be none too profitable.

Resuming the enumeration of the existing channels in the Port of New York, the Newark Bay Channel is 20 ft. deep and 400 ft. wide, with a side channel of similar dimensions connecting the central channel with the Port of New York development. At present a project is under consideration for deepening this channel to 30 ft. Some difficulty has been experienced in deciding as to the best policy regarding this particular channel because of the obstructive bridge near its foot, connecting the mainland with the Bayonne

Peninsula. This bridge, belonging to the Central Railroad of New Jersey, is now required to be reconstructed under the existing project, but as yet no change has been made, and it is perhaps worth saying that until it is positively known that this bridge will be changed to permit the easy passage of deep-sea carriers, it would be useless to spend any money in deepening the main Newark Bay Channel.

The remaining important channel is the Arthur Kill Channel, which is 25 ft. deep and 400 ft. wide. This channel is a very busy waterway and, recently, the traffic has been so heavy as to call for a re-examination of its needs, and the War Department has recommended to Congress that a project for a channel 30 ft. deep and 400 ft. wide through Raritan Bay and Arthur Kill, connecting with the corresponding channel in Kill van Kull, be adopted at a total cost estimated one year ago at about $10 400 000.

This summarizes the situation as to the main channels in the Port of New York. The War Department and the Corps of Engineers constantly study and observe traffic and terminal conditions in order that they may be able to appraise intelligently the needs of the ports. Whenever these studies indicate the desirability of greater channel accommodations, either by deepening and widening existing channels, or by extending them and creating new ones, the need will certainly be reported to Congress in ample time to avoid the damage or inconvenience that might result from a failure to take timely action. So far as is consistent with the duty of protecting the National Treasury from unprofitable or unnecessary expenditures, the speaker can safely promise that the War Department will always, in the future as in the past, deal not only considerately but liberally with shipping and with water-borne traffic. When their needs are in the form of an extension of any channel so that additional terminals may be created to supplement existing terminals already fully used, no difficulty will be experienced as far as the Department is concerned in securing a recommendation to Congress that the channels be appropriately enlarged. It is necessary, of course, to safeguard this promise by the statement that its policy is to insist that proper use be made of the portions of the shores of any port to which access has been afforded by the creation of adequate channels, and this insistence may well take the form of requiring that where important sites are occupied by activities not located by necessity on the water-front, these sites be vacated and suitable terminals provided there before the Government is forced to further expense in the provision of channels. This weakness in the development of the Port of New York is well known and is emphasized in the monumental report of the Joint Commission in which attention is repeatedly drawn to the fact that, especially on the two sides of the Hudson River, the shores are used as railroad stations and not for transfer from rail to ship and ship to rail. This evil is especially noticeable on the west front of the Hudson, where practically all the Jersey City front and much of that of Hoboken and Weehawken is so used. What a relief it would be to the Port of New York if these considerable lengths of shore line were available for the construction of modern piers and their adjuncts. It ought not to be regarded, therefore, as unreasonable for the National Government to insist, as was recently done in the case of a South Atlantic port, that

all available frontage be properly utilized before extensions of channels are made.

Reverting now to the relation of the War Department to the National port terminal problem, it is probably well known that, in recent years, Congress on frequent occasions, has indicated its view that port and river terminals were essential adjuncts of waterway improvements and that they should be incorporated in any proper plan of such improvements and required to be supplied by the local interests. The extent of the preoccupation of Congress with terminal matters and the importance which it attaches to the creation of proper terminal facilities is shown by the legislation incorporated in Section 7 of the River and Harbor Act, approved July 18th, 1918, which reads as follows:

"Section 7. That hereafter the Chief of Engineers, United States Army, shall indicate in his annual reports the character of the terminal and transfer facilities existing on every harbor or waterway under maintenance or improvement by the United States, and state whether they are considered adequate for existing commerce. He shall also submit one or more special reports on this subject, as soon as possible, including, among other things, the following:

"(*a*) A brief description of such water terminals, including location and the suitability of such terminals to the existing traffic conditions, and whether such terminals are publicly or privately owned, and the terms and conditions under which they may be subjected to public use.

"(*b*) Whether such water terminals are connected by a belt or spur line of railroad with all the railroads serving the same territory or municipality, and whether such connecting railroad is owned by the public and the conditions upon which the same may be used, and also whether there is an interchange of traffic between the water carriers and the railroad or railroads as to such traffic which is carried partly by rail and partly by water to its destination, and also whether improved and adequate highways have been constructed connecting such water terminal with the other lines of highways.

"(*c*) If no water terminals have been constructed by the municipality or other existing public agency there shall be included in his report an expression of opinion in general terms as to the necessity, number, and appropriate location of such a terminal or terminals.

"(*d*) An investigation of the general subject of water terminals, with descriptions and general plans of terminals of appropriate types and construction for the harbors and waterways of the United States suitable for various commercial purposes and adapted to the varying conditions of tides, floods, and other physical characteristics."

A further expression of the policy of Congress in regard to terminals is contained in the River and Harbor Act approved March 2d, 1919, in which the following statement is incorporated:

"It is hereby declared to be the policy of the Congress that water terminals are essential at all cities and towns located upon harbors or navigable waterways and that at least one public terminal should exist, constructed, owned, and regulated by the municipality, or other public agency of the State and open to the use of all on equal terms, and with the view of carrying out this policy to the fullest possible extent the Secretary of War is hereby vested with the discretion to withhold, unless the public interests would seriously suffer by delay, monies appropriated in this Act for new projects adopted herein, or for the further improvement of existing projects if, in his opinion, no water terminals exist adequate for the traffic and open to all on equal terms, or

unless satisfactory assurances are received that local or other interests will provide such adequate terminal or terminals. The Secretary of War, through the Chief of Engineers, shall give full publicity, as far as may be practicable, to this provision."

It will be noted that the Chief of Engineers is required to report annually on the character of the terminals existing in every harbor or waterway under improvement by the United States, and to express an opinion in regard to their adequacy. This requirement, of course, cannot be fulfilled unless consideration is given not only to the character and number of the terminals themselves, but also to the place of each port in the National transportation system, its tributary territory, the traffic it ought to serve, its probable future growth, and the effects, restrictive or otherwise, of existing channel developments. These questions are so important and far reaching that to answer them satisfactorily it has been found necessary to expand greatly the functions of the Board of Engineers for Rivers and Harbors which, until about two years ago, had for its main function the review of projects, active or proposed, for river and harbor improvements. At present, therefore, this Board, under a plan approved by the speaker, is making a close and intensive study of every important seaport in the United States and an analysis of its most useful place in a properly conceived National system. These investigations permit replies with considerable certainty to questions relating to the adequacy of port terminal development.

The special reports called for in Paragraphs *a*, *b*, and *c* of the 1918 legislation have been consolidated in a single report, which is being printed as House Document No. 652, 66th Congress, 2d Session. This bulky volume is in greater part a compilation of facts relating to all water terminals on waterways now being improved or maintained by the United States. Its principal service to the public, and especially to engineers, is to place before them in authoritative form the many things relating to existing port improvements, which usually are known only in a general way, if at all. For example, it gives to every careful reader a serviceable idea regarding the state of development concerning any port in which he may be interested.

The investigation of the general subject of water terminals, called for in Paragraph *d* of the 1918 legislation, has been made by the Board of Engineers for Rivers and Harbors, and the results are contained in a comprehensive report by Capt. F. T. Chambers, U. S. N., associated with the Board as expert adviser on terminal matters. This report is now in the press as House Document No. 109, 67th Congress, 1st Session. The illustrations accompanying the report are numerous and afford examples of nearly every useful type of terminal known to exist. In order to show the scope of this report, the speaker will enumerate the headings, as follows:

Ocean terminals are considered under the following sub-divisions: sheltered location necessary; ports; terminal facilities; tides and floods; anchorage; fore and aft moorings; locks and basins; docks, quays, wharves, and piers; open or uncovered terminals; ground storage for materials in bulk; ocean and inland waterways terminals; other open or uncovered terminals; naval stores; covered terminals for material in bulk; storage for liquor material in bulk; fire protection; grain elevators; covered terminals for miscellaneous or package freight; types of construction; piers; materials for construction;

examples of pier construction; quays and wharves; warehouses; warehouses for special purposes; railroads; lighterage; mechanical equipment; ship's gear; ship's gear in combination with portable winches and cargo masts; wharf cranes; types of cranes; monorails and telphers; conveyors; slot conveyors; pan and bucket conveyors; chain hook conveyors; portable chain and belt units; gravity conveyors; gravity roller conveyors; piling; machines; fixed elevators; trucks; self-propelling vehicles; tractors and trailers; floating equipment; floating derricks; steam lighters; suction grain elevators; chain belt bucket units for grain; bunkering equipment; portable bunkering units; car floats; repairs and reconditioning; types of dry docks; seaport terminal; planning, including harbors, channels, ship characteristics, tides, available water-front, character of exports and imports, open and covered wharfage, railways, piers, and slips, floor space under cover, railway trackage at water-side, hoisting equipment, railway tracks and roadway at rear of shed, railway yards, disposal of dredged material, quay or wharf construction storehouses, mechanical equipment, justification for transit sheds of large area, port terminal charges; port terminal control; forms of port control, public, private and railroad, railway port terminal charges, public management.

Inland waterway terminals are considered under the following subdivisions: canals and rivers; navigable depths and variation of water level; terminals for waterways of constant level; terminals for handling bulk cargo; terminals for rivers of wide variation of water level.

The 1919 legislation as to the essentiality of having at least one public terminal at each waterway locality under improvement by the United States, is a plain declaration of a belief which has long been held in public places, though not so formally expressed. The idea is sound, but goes hardly far enough. Every one who has come in contact with the port terminal problem sooner or later has been forced to conclude that perhaps the greatest evil in the situation and one of the principal "National Port Problems" is the undue extent to which the water-fronts of the principal seaports have passed into railroad control or ownership. The effect of such railroad predominance has often been to influence the prosperity of the seaports, some of which have been favored at the expense of others. The situation in this regard is quite fully covered by the Board of Engineers for Rivers and Harbors, in a recent report which relates to the South Atlantic and Gulf Ports, and recommends steps to prevent the railroads from continuing this kind of activity, by compelling them to make rates for terminal services fairly compensatory for the work involved and the use of the facilities concerned. The belief of the Board is, however, that the railroads should be compelled to divest themselves of their ownership of these port terminal properties and that preferably they should pass to public ownership and control.

It may be of interest to quote the closing paragraphs of the report on the investigation of terminal charges at South Atlantic and Gulf Ports by the Board of Engineers for Rivers and Harbors:

"In promoting water transportation, the War Department and the Shipping Board recognize that a primary necessity is the provision of adequate terminals, and they have an interest in securing such changes in existing conditions as may be necessary to establish them. The absorption of terminal charges in the rate for the line haul and their inadequacy render it impracticable for the private terminal to compete with the railroad terminal for through business just as a large mercantile establishment by selling one commodity below cost can destroy the business of a special dealer in that commodity, and still make

a profit on the total business. It is well known that except at a few places where unusual conditions prevail, private terminals cannot be operated at a profit. The situation is somewhat different in respect to publicly owned terminals. Municipal corporations generally have good credit and are able to borrow money at low rates. Even though public terminals may show an apparent deficit, it may well happen that they will cause such growth in general prosperity and in the volume of business as will outweigh any tax burden caused by the terminals.

"To permit the free flow of commerce through our ports, the obstacles in the way of creating modern terminals must be removed, and since the carriers themselves cannot be expected to initiate the necessary reforms, prompt steps should be taken by the United States. Two remedies have been suggested: First, a scale of terminal charges sufficient to cover the actual cost of the service, plus a reasonable return on the investment. This would enable private terminals to operate if the railroads were obliged to pay them such charges for services actually performed. Second, the discontinuance of the practice of absorbing terminal charges in the rate for the haul. The latter remedy has been widely advocated, but clearly presents some difficulties in its application, and necessarily implies a complete revision of all rates to and from water points.

"In some cases an effective remedy would be the ownership and operation by the State or municipality of all water terminals used for public transportation purposes, including a belt-line railroad affording connection with all wharves and with all railroads serving the port. With the switching, wharfage, handling, and storage charges, in the hands of the State or municipality, every terminal within the port might expect equal treatment, and the responsibility for providing adequate facilities would rest squarely upon the community itself. This remedy is not practicable at many localities, however, owing to the extensive occupation of the water-front by private interests.

"The difficulties confronting the solution of the problems herein considered are fully recognized, and it is obvious that decision as to the wisdom of any plan for correcting existing conditions should be reached only after the most searching investigation of its effect upon the movement of traffic and the relationship between competing ports. The matter is of such importance to the commerce of the country and the success of the merchant marine, however, that no difficulties should be allowed to stand in the way of securing an adequate remedy."

On March 25th, 1921, the Secretary of War brought this situation as to inadequate or unfair terminal charges to the attention of the Interstate Commerce Commission. As this letter is an impartial summary of the situation, it seems of sufficient interest to be read in its entirety, and is as follows:

"By reason of the war emergency the country has been provided with a large commercial fleet upon which a very large sum already has been expended. It is indispensable to our safety and prosperity as a nation that this fleet operate successfully and economically in competition with the vessels of our maritime and commercial rivals.

"Indirect and direct charges of a ship in port run from $1 000 to $5 000 per diem, and every day spent by each ship in port in excess of the minimum time required to load and discharge cargo, and to take on fuel and supplies, is a waste, which summed up for the entire fleet amounts under present conditions to millions of dollars per annum. Ships earn money only when they are kept moving with profitable cargo. Efficient operation, therefore, requires that the days spent in port be the fewest practicable.

"With the object of promoting, encouraging, and developing ports and transportation facilities in connection with water commerce, Section 8 of the Merchant Marine Act authorizes the Shipping Board, in co-operation with

the Secretary of War, to investigate territorial regions and zones tributary to ports, in order to determine what rates, charges, rules, or regulations should be established, and, in case changes are found necessary, to submit its findings to the Interstate Commerce Commission for such action as the Commission may consider proper under existing law. The Board of Engineers for Rivers and Harbors on behalf of the War Department has therefore been investigating the subject of port facilities, and both the Chief of Engineers and the Board are convinced that the success of our commercial fleet is largely dependent upon the economical operation of our ports.

"The results of the above investigation have been compiled by the Statistical Section of the Board of Engineers for Rivers and Harbors and are shown in the attached paper, which also contains excerpts from the various laws assigning to the War Department and the Shipping Board duties in connection with port regulation and port development. In view of the evidence presented in this compilation, it is requested that the subject of port charges and practices at South Atlantic and Gulf ports be considered with a view to the initiation of the necessary reforms, so that commerce may flow through our ports with the greatest possible speed and the least friction; or, in other terms, at the least expense to both the shipper and the ship owner.

"The Chief of Engineers points out that there are three forms of terminal control. These are (*a*) public (State or municipal); (*b*) railroad; and (*c*) private; and at none of our important ports is the control absolute under any one of these three heads. San Francisco and New Orleans have the nearest approach to complete public control, but at all of our important ports there is some railroad control, and at many of them there are also private terminals. Under present conditions of railroad operation, the railroads have no direct incentive to build or to operate terminals such as would turn the ships around in the least practicable time, and, even should such incentive exist, they are not now financially able to provide the costly wide piers and the expensive mechanical equipment necessary to the greatest economy. As will be seen from the attached manuscript, the tariff charges for handling cargo over the railroad terminals at practically all of our South Atlantic and Gulf ports are from one-quarter to one-half the actual cost of performing the work and this is the amount that the railroads allow private terminals for such service performed by them. Under such circumstances private terminals now in existence do mostly a warehousing business, or handle local freight only. They cannot exist on the railroad divisions for through overseas business. Private capital cannot therefore be attracted to invest in such terminals, nor can the largest cities be expected to construct modern terminals, if, as sometimes happens, the situation created by the railroads is such that it is certain in advance that an adequate return cannot be had with which to amortize the bonded debt and that the indirect gains are so seriously restricted as not to counterbalance this direct loss.

"Obviously, it would be advantageous to the public to have full freedom in the choice of routes over which its commodities may be transported. Such choice is now, however, limited and one of the limiting conditions is the fact that the route that otherwise might be preferable may have at the terminal port facilities such as wharves, transit sheds, storage warehouses, and the like whose capacity is merely sufficient to care for existing business or whose equipment may be poor, antiquated, inefficient, or uneconomical. Such restraints upon the adoption of the economic path for moving traffic should, in the general interest, be removed and the opportunity should be afforded all concerned to create, with reasonable prospect of adequate return, such port terminal facilities as seem needed and desirable. The only way in which this can be brought about is to put all such terminals upon a proper basis. The difficulties of the problem are no doubt considerable, but it is believed that this can be done, in some cases by revising the terminal tariffs so as to be reason-

ably compensatory for the service actually performed, thereby permitting privately owned terminals to be created and operated. In general, however, the ownership and operation of all terminals by the local public authorities or by the State constitute the best solution and this course should be adopted wherever possible.

"I would ask that the facts herein presented be considered with a view to the early application of such remedies as may promote the end in view, namely, the reduction to a minimum of the cost of transporting the goods that enter into our water-borne trade, both foreign and domestic."

As a result of these representations the Interstate Commerce Commission will soon hold a series of hearings at all the ports included in the investigation; the first hearing is to take place at Norfolk, Va., on September 19th, 1921, and the others will follow in rapid succession. It is not unreasonable to expect that the final outcome will be to place the port terminal situation as to charges on a sounder basis.

It will be observed that the War Department and the Corps of Engineers believe it to be for the best interest of the public that water-front improvements be publicly owned and operated, but in this matter of public operation it is important that caution be observed not to permit public operation by the political agencies sometimes utilized by city and State governments, which would be to the serious detriment of seaport terminals. The form of public operation which the War Department favors and which the speaker strongly advocates is that under which some form of port authority exercises control on behalf of the public, similar to the English so-called "Port or Harbor Trust." This body is dominated by representatives of the shipping, commercial, and business interests of the port and the purely political members constitute a small minority.

One of the greatest obstacles to the progress of American shipping business and to the promotion of the export and import traffic is the lack of unity of management. In this respect, Americans should take a leaf from the book of their British competitors, who have made this subject a study for many years of successful maritime business. Every considerable British port has a really unified management.

Forms of Port Control.

In general, it may be said that port terminal control is of three kinds: (1) public; (2) railroad; and (3) private.

(1).—Public Control.—This is best illustrated by the practice of such ports as Hamburg, which is a commercial State, and of London, Liverpool, and Bristol, England, which more nearly represent what we should endeavor to attain in the United States. The Port of London suffered for many years from the internal competition of a number of separate dock companies within the port. It was not until July, 1908, that the entire port property was taken over and put under the London Port Authority, competition within the port thus eliminated, and the port as a whole put in a position to direct its energies to competition with other ports. This Port Authority is called in England a "Trust", not as this word is generally used in the United States, but in the sense of a corporation operating without profit for the public benefit. This does not mean that port charges are not assessed, but only that the revenues

obtained from such charges are devoted exclusively to the interest of the port terminal, and not, as in the City of New York, to such extraneous matters as the cleaning of the city streets and other items of city maintenance. In other words, where New York devotes a considerable part of its revenue from the city-owned piers to general city purposes, London devotes its tolls to that part of the welfare of the community which is bound up in the promotion of its shipping trade. Another organization called "The Trinity House" is responsible for the lighting and buoying of the river. The Metropolitan Police are the guardians of the port, and the Corporation of the City of London supervises the sanitary conditions with regard to shipping.

The Port Authority is empowered to collect tolls on ships and goods, and has full jurisdiction over the various docks and terminals. It is constituted as follows: Appointed members, 10: one by the Admiralty; two by the Board of Trade; two by the London County Council (being members of the Council); two by the London County Council (not being members of the Council); one by the City Corporation (being a member of the Corporation); one by the City Corporation (not being a member of the Corporation); one by Trinity House. The elected members number 18: Seventeen by the payers of dues, wharfingers, and owners of river craft; and one by the wharfingers. In addition, the Chairman and Vice Chairman may be appointed from outside the membership of the Authority, making the total possible number 30.

The organization at Liverpool is known as the Mersey Dock and Harbor Board. It is quite similar in its constitution to the London Authority, and has 28 members, 24 of whom are elected by the dock rate-payers, that is, persons paying rates and dues on ships and goods only, the remaining 4 being appointed by the Mersey Conservancy Commission, which consists of the First Lord of the Admiralty, the Chancellor of the Duchy of Lancaster, and the President of the Board of Trade. The Board of Trade, unlike the Boards of Trade in the United States, is a National institution corresponding in many respects with the Department of Commerce. In making appointments to the Mersey Board, the effort is made to give the various trading interests in the local shipping community a proportionate representation, and at the present time this is said to work well. Such organizations as those at London and Liverpool may seem to the casual reader unwieldy, but each organization is divided into separate working committees with apparently good effect.

The Port of Bristol is owned and operated by the Municipal Corporation, which thus has jurisdiction over the entire dock system, the control being vested in the City Council, which, for purposes of administration, appoints a sub-committee. This committee employs the executive officials and gives them a free hand in the management. It is declared that politics plays no part in this administration.

(*2*).—*Railroad Control.*—Some of the ports of England, notably, that of Immingham, are entirely in the hands of the railways. The port terminal is operated as a part of the railway system, and as a means of getting business. In fact, many ports in the United States are operated on this same railway basis, the port terminal charges being nothing, or nominal, wherever the railway is enabled to get business by offering dockage to the ship.

(3).—*Private Control.*—Terminals of this class are operated by private individuals or corporations in the pursuit of their own business in all parts of the world. They may be adapted to the handling of only one product, or to miscellaneous freight. The manufacturing concern may be located at the terminal, and may use its water-front facilities in direct connection with its business only, or, in some instances, may conduct a separate terminal and warehousing business.

General.—It may be said in general that taken as a whole no port in the United States comes under any one of the three previously named headings. In all ports of any size all three methods of administration are in force. It is true that at San Francisco and New Orleans public port authorities are controlled by the State, but, at San Francisco, the Harbor Board confines its activities to the water-front at San Francisco, while the railways have terminals across the Bay at Oakland, on which side of the harbor there are also private terminals. At New Orleans, also, the State Board and the railways have separate terminals.

In some ports it has been the practice to lease the publicly owned terminals to private operators. Although there may be some financial justification for this policy, it seems well to observe some degree of caution in granting exclusive and particularly long-time leases for the use of piers and wharves. The recent experience has been that such exclusive leases may result in less efficient use of the terminals than is desirable for the best interests of the port. A lessee, for example, may keep his pier idle rather than permit its use by an actual or potential competitor, or he may make such high charges as to drive business away from the port. This actually happened in New York during the World War. If leases must be made, they should preferably be in the form of a first call on berth, leaving to the public port authority the right, when the berth is vacant, to assign to it vessels other than those belonging to the lessee.

At the same time, the prestige of a port depends greatly on the number of regular services that it affords. Liners belonging to such services are most readily identified by the public when their sailings take place from definitely known piers. The system of first call on a berth usually meets this situation adequately, but if it is necessary to promote liner service a limited number of concessions from the rule might be made.

It seems well to repeat the result of the review of the port situation in the United States, as stated in the report of Capt. Chambers, to which reference has previously been made. This review shows:

1.—That there is not, in most ports, a well co-ordinated management and that a well constituted port authority is the first need.

2.—That under these port authorities, comprehensive plans should be evolved, based on principles hereinbefore laid down.

3.—That a port can only be successful when this plan is based on the business available in its tributary area and brings ample railway facilities into the closest practicable juxtaposition with the water-front, with sufficiently wide areas available for cargo classification.

4.—That, with the increased cost of labor, mechanical means, wherever practicable, should be adopted for handling goods, and that such means should be at hand for every kind and shape of package.

5.—That ample railroad tracks should be available close to the terminal for car storage and car classification.

6.—That ample warehouse capacity should be provided, in order that both ships and cars may be dispatched in the shortest practicable time. Too many American ports are lacking in warehouse facilities, or have such facilities at an inordinate distance from the terminal, thus involving cartage, or extra handling, for local railroad haul.

7.—Where cartage is necessarily a feature in the port business, roadways and loading platforms should be provided for the full accommodation of trucks.

8.—That bunkering facilities of such character as to supply the necessary fuel to the ship while handling cargo should be available.

9.—That ample repair and dry-docking facilities should be provided.

No matter how ample the provision of terminal piers, sheds, railways, and mechanical equipment, a port will only succeed under an efficient and well co-ordinated management.

Although this discussion of the general subject of "Terminals" has been from the point of view of the War Department, the speaker feels that in closing he should state that it is the policy and the wish of the Department to be helpful to the people it serves, and in the discharge of the trust imposed and in using the discretion confided by law, the desire is to be liberal and not narrowly technical. Therefore, an invitation is extended for the frank and full discussion of problems, with the promise that they will receive patient and sympathetic consideration.

DEVELOPMENT OF THE SMALLER PORTS

By Frederic H. Fay,* M. Am. Soc. C. E.

As a nation, Americans are slowly coming to a realization of the economic value of their seaports to the country, a lesson learned long ago by their European neighbors and more recently by their Canadian cousins who have been more alert than themselves and by whose example and foresight they may well profit.

Before the World War it was commonly stated that a ton of freight could be hauled 5 miles by water as cheaply as it could be carried 1 mile by rail. With changed conditions, due to the war, this ratio may now be changed. Leaving aside, however, for the moment, the cost of loading and unloading which admittedly constitutes a large part of a transportation charge, it is to-day unquestionably true that freight can be hauled at a given cost a far greater distance by water than by rail. As a nation, Americans should be giving more serious consideration to the importance of water-borne, coastwise traffic, and since the opening of the Panama Canal, particularly to the possibility of greatly increasing coastwise traffic between the Atlantic and Pacific ports. A full development of this domestic commerce by water routes between these two coasts should result in extensive changes in the routing of freight by rail and in the rail-rate structure, stimulate the interchange of commodities between remote sections of the country, and prove of economic benefit to the nation as a whole. Although only in its infancy, the small beginnings which in these abnormal times have already been started in the Atlantic-Pacific coastwise trade, have had a marked effect in certain directions. Low coastwise rates have been established which, in some instances, are less than half the transcontinental rail rates, and the results thus far give promise of the larger results to follow when business has returned to normal conditions.

Until recently the development of American ports has been left very largely to private interests, usually to the railroads which naturally have made their developments with a view to their own immediate gains and without regard to the larger advantages to the public as a whole through the unification and co-ordination of the development of a port as a single unit. The American public is gradually awakening to an appreciation of the importance of public ownership or control of port facilities and of the commercial value of ports, especially as agencies in foreign commerce. Efforts toward public port development should not be concentrated on a few of the larger ports and primarily for purposes of foreign commerce; public attention should also be drawn to the benefits to be gained by the development of the smaller ports as well, not having in view foreign commerce alone, but also the need of stimulating water-borne domestic commerce and the economies and benefits resulting therefrom. That part of the Engineering Profession concerned in port-development work has a public duty to perform in giving serious study to the situation as it exists at the smaller ports and of acquainting the public with the facts as they find them.

* Boston, Mass.

Economic Value of the Seaports.

A community located on a busy harbor enjoys many special advantages. Marine carriers make large capital outlays in connection with their terminal operations. They buy coal, provisions, and other supplies from local dealers; pay substantial sums in wages to longshoremen and freight-handlers; spend money for repairs and employ towing companies and pilots. Even at many of the smaller ports millions of dollars attributable to port activities alone are distributed annually in the port community.

In a larger way, however, the port is the servant of the interior. For every ton of local freight shipped to and from the port community itself, there are usually many tons shipped to and from the district immediately tributary as well as from more remote inland territory. This indicates the true relationship between the seaport and its hinterland. The port serves the interior in the latter's dealings with coastwise and foreign ports, and provides the smoothest mechanical and commercial means for the movement of inland freight to and on the water. Its development calls into life new water lines and betters the service of existing lines, and its merchants find new markets for inland products.

A well-developed port is of indirect benefit to city, State, and nation, and its development should not be left in private hands, but should be under public control and participated in jointly by municipal, State, and Federal governments.

The principle of Federal aid has long been established through the assumption by the Federal Government of the work of dredging and improving harbor channels, harbor protection, and the provision of aids to navigation.

More recently State agencies have undertaken the building and development of water-front terminals, the reclamation of water-front lands, and even the dredging of harbor channels to supplement the improvements made by the Federal Government.

In a few instances municipalities, at their own expense, have provided extensive water-front terminals; in others, they have co-operated with State agencies in such undertakings, as, for example, in acquiring water-front sites on which State funds are spent for pier construction.

The public is coming to a general recognition of the soundness of the principle that the seaports are an important National asset and, as such should be under public, not private, control; and, further, that Federal aid should properly be given to the improvement of the water facilities of a harbor, while States and municipalities may properly work together in providing water-side facilities and necessary improvements on land.

Requisites of a Port.

It may not be out of place to consider briefly the chief requisites of a port since the engineer must have these fundamentals in mind when studying the problems of any port, large or small.

Water Facilities.—There must be a good harbor, well sheltered and free from ice; a natural harbor, if possible, or, at least, a location such that an harbor may be developed at moderate expense. The harbor should have

natural deep-water channels, or else a bottom such that adequate channels may be dredged and maintained at reasonable costs, and should preferably be in close proximity to the open ocean with a safe and direct approach channel and, if possible, an absence of an outer bar.

Rail Facilities.—On land, there should be adequate railroad facilities, not only locally throughout the district immediately tributary to the port, but also trunk-line connections to more remote points in the interior of the country. At the port itself, there should be adequate railroad terminals, including classification and storage yards and supporting yards in close proximity to the water-front piers. Important for the success of a port are belt-line rail facilities for the interchange of freight between the several roads and for bringing freight from each railroad to each pier, and fortunate is the port where such facilities exist or can be provided at reasonable expense.

Pier Facilities.—The pier is the connecting link between rail and water carriers. Piers should have transit sheds sufficiently large for the assembly and rapid handling of cargoes. Present-day ships call for piers of large size and larger shed capacity than those commonly built in the past. There is general agreement as to the importance of railroad connections to piers, but many port authorities, shippers, and steamship officials in the United States have been slow to grasp the importance of equipping piers with adequate, modern, mechanical apparatus for the rapid handling of freight.

In addition to the general factors already noted, recognition should be given to the importance of good highways throughout the territory immediately tributary to the port. The motor truck has its legitimate field in the inter-urban transportation of high-class package freight, and by its use many inland industrial communities may have connection with the seaport more cheaply than by rail.

It is obvious that a port situated at the mouth of a navigable stream has the added advantage of water transportation as well as rail connection with its hinterland.

Where advantages such as those already outlined exist, public authorities should give serious consideration to the possibilities of their proper utilization, and this is particularly true of many of the smaller ports. Europeans have realized more keenly than Americans the importance of access to the sea, and in many instances enormous sums have been spent in overcoming natural obstacles and in making seaports out of inland, or semi-inland, cities, as at Manchester, Glasgow, Amsterdam, Rotterdam, and Hamburg.

It is in connection with the smaller ports particularly that the engineer should live up to his duty as an economist, study each local situation with critical analysis and breadth of vision, and be a pioneer in calling public attention to the possibilities that lie before us.

Danger of Concentration in a Single Large Port.

In the United States the tendency has been to foster the development of the Port of New York which, for years, has handled approximately 50% of the country's foreign commerce, and ignore the natural advantages of, and the importance of developing, many of the smaller ports. Americans may well

ask themselves the question whether it is a wise policy for the country as a whole to have so large a proportion of its commerce tied to a single port.

Less than two years ago, the Collector of the Port wrote of conditions at New York* as follows:

"We are accustomed to speak of New York as the greatest commercial port in the world. So it is. To-day its water-borne commerce, foreign and coastwise, is greater than that of any other two ports in the world; but that is no reason why it will continue to hold this place for all time. * * * When the pressure of war traffic came upon us the port of New York 'broke down', or so nearly broke down that it was a matter of National distress and alarm. Along the extensive shore line of the inner harbor there was ample space for handling all the traffic the war sent to our port; but only a part of the harbor shore line was available. There were not piers enough to accommodate the vessels, and most of the available piers were too small, inaccessible, improperly equipped, or being used for some purpose necessary for the domestic demands of the city."

In war time, the successful blockading of this one port by a strong enemy fleet would be a tremendous handicap. At any time, local conditions may precipitate a longshoreman's or freight-handler's strike with paralysis of shipping, as New York well knows by recent experiences. Only last winter (1920–21) the influx of immigrants at New York was so great that the immigration and quarantine facilities of the port were unable to handle them, and it was necessary to divert ships to other ports to land such passengers. The cost of handling freight is enhanced by lack of belt-line railroad facilities, by the necessity for lighterage to supplement the roads, and by lack of modern piers equipped with modern freight-handling facilities. Furthermore, the values of New York water-front property are so great as to place a substantial over-head burden in the carrying charges on the investment in water-front terminals. All these factors tend to enhance the cost of handling freight in New York and the question may well be asked whether a considerable proportion of the business now passing through that port could not be handled cheaper, with consequent economic saving to the nation, at some of the smaller ports.

Efficiency and economy must be applied to the nation's business if it is to take its full part in world affairs.

Advantages of Development of Smaller Ports.

Instead of putting so many "eggs in a single basket", which may come to smash at any time, as has been proven at New York during and immediately after the World War, is it not better to get a diversion of commerce through other ports possessing good, if not equal, natural advantages, and to awaken in the public mind a realization of the National, State, and local benefits, arising from suitable public development of some of the smaller ports?

The development of smaller ports means a diminution of labor troubles and less likelihood of strikes having such serious effects on water-borne commerce. Such development means not only the building up of cities where the ports are located, but also the districts tributary thereto and a more even

* "New York's Endangered Commercial Leadership," by Byron R. Newton, Collector of the Port of New York, *The Street*, December 31st, 1919.

distribution of prosperity throughout the country at large. Americans have been too much inclined to consider port developments only in the light of foreign commerce, and have ignored the advantages of port development in increasing and cheapening domestic commerce and in lessening the cost of living. In the smaller ports, the cost of development is substantially less than the provision of equally good facilities in a large port like New York. Water-front properties are relatively cheap. Belt-line rail connections are usually more readily provided, if they do not already exist.

To-day, large industries are coming to realize the disadvantages of concentration in the larger cities and the advantages to be gained by locating their plants in smaller communities where better living and labor conditions are found. The development of smaller ports to cheapen transportation to and from smaller communities, both on the seaboard and in the district tributary to the port, is of direct concern to industries which are seeking to develop plants in the smaller cities and towns. This tendency toward distribution rather than concentration is a helpful one for industries and the country as a whole, and makes necessary more than before serious consideration of the need of development of smaller ports to handle not especially overseas traffic, but primarily coastwise traffic.

Port Development at Public *versus* Private Expense.

The proper development of American ports is to-day a vital problem. With only a few exceptions, ports in the United States were originally developed by railway or other private interests, and there is ample proof that, in most cases, the individual interests have failed properly to develop and co-ordinate the water-front facilities. Usually, the development has been narrow and selfish, and each railroad has naturally looked after its own individual interests. The recent trend of development has demonstrated plainly that the port must be considered as a whole, with the railroad and other terminal facilities co-ordinated so that they develop the port and the territory tributary to it in the most economical manner and on the broadest possible scale.

Where port development has been by railroad agency alone, it has naturally progressed slowly and, in general, only with the assurance that it would prove to be a paying investment. The tendency until recently has been for each railway to attempt to secure for itself the most advantageous location for its wharves and docks, and then to throttle the development of the port as a whole by setting up artificial barriers through the medium of switching charges, absorption of wharfage and handling charges, etc. As a result, many ports have become in reality a collection of several small ports, each serving as a terminus for an individual railroad, interchange of traffic between various parts of a harbor being rendered both slow and expensive.

A study of North American ports where such facilities are publicly owned or controlled shows that such ports have been a success in every case and this is obviously true for some of the notable ports of Europe. Not only has the operation of these terminals proven the investment to be sound, but the ports have been distinctly benefited by public control. Discriminative railroad practices have been eliminated, more flexibility of operation has been assured,

and commerce has been increased, due in large measure to the greater opportunities for expeditious and cheap handling of traffic.

No private party can be expected to make the large expenditures required for adequate port facilities. These must be made by the public, since States and cities can command a lower rate of interest and will benefit not only from a direct return, but even more largely from the indirect return which increases the business and the prosperity of the city and State in which the port is located. Private parties must consider the cost of water-front terminals purely from the standpoint of direct and immediate financial return, sufficient not only to pay carrying charges, including depreciation, but to return a reasonable profit to the investors. On the other hand, the community may very properly find the greater part of its profits in indirect returns, such as the prevention of the port from being throttled by its more active competitors, increase in commerce, and the promotion of the welfare and prosperity of the city and of the State at large.

Portland, Me., as Typical of the Smaller Ports.

Portland, Me., is a good example of one of the smaller but important ports the development of which has long been neglected, but where public sentiment has been aroused. The needs have been recognized, and the State and the municipality are co-operating toward a modern development. Portland possesses a harbor which, from the standpoint of natural advantages, is one of the best of American ports. Although not comparable with New York in size, it is of ample area with a natural deep-water channel and with no bars at its entrance. The harbor is well sheltered, in close proximity to the open ocean, with a channel so direct that steamships making regular calls dispense entirely with pilots and enter the harbor at any time of day or night, at any season of the year, under the direction of their own officers.

Portland is the Atlantic terminus of the Grand Trunk Railway and is the natural winter port for the Dominion of Canada. It is also a terminus of the Boston and Maine and the Maine Central Railroads, and is the nearest port in the United States to the United Kingdom and Europe. That the Federal Government has considered it to be one of the important Atlantic ports is shown by the fact that the harbor has been heavily fortified. During the World War, troops and supplies were shipped therefrom to the full limit of the existing water-front terminals.

Up to the present time, the only piers accommodating overseas shipping were those of the Grand Trunk Terminal; and, except for certain other wharves owned mostly by railroads and equipped for handling such bulk freight as coal, china clay, and sulphur, the wharves are obsolete and relics of the days of sailing ships when Portland had an extensive trade with the West Indies.

Recently, however, the need of increased water-front terminal facilities has been keenly felt. Certain steamship lines which have sought to establish themselves have been unable to do so on account of the lack of facilities for the accommodation of their ships. Only last winter (1920-21), the North Atlantic and Western Steamship Company, the boats of which, engaging in

the Atlantic-Pacific coastwise trade, had been berthed during the summer at the piers of the Grand Trunk Railway Company, found great difficulty in securing accommodation during the winter months, during which period the Grand Trunk Terminal is used to capacity, and succeeded in placing only a few boats at one of the Railway Company's piers through special arrangement with that road and because the Company's business last winter was not at a maximum.

Through the instigation of the Portland Chamber of Commerce, an agitation was begun which has resulted in the starting of further development through co-operation of city and State. The city has just provided the site of the first of a series of publicly owned piers, and the State is now about to construct such a pier alongside the Grand Trunk Terminal. Unlike many other ports, Portland already possesses a belt-line railroad connecting all the railroads entering the city, and the new State pier is so located on this belt line that freight at the pier will be handled on equal terms to and from the railroads. The State and the city are jointly embarking on a policy of port development with the idea not only of providing facilities for increased coastwise business, but also of increasing overseas commerce. The Maine Central Railroad connects with the Canadian Pacific Railway of Canada, which road it is understood has for some time desired entrance into Portland, and this entrance will be assisted by the building of the State-owned terminal.

Portland possesses a situation which is unique among the North Atlantic ports in that through the Grand Trunk and Canadian Pacific Lines, it is a natural outlet, especially in the winter months, for territory extending as far as the Canadian Northwest; and because of the United States connections of these two roads, it is a logical outlet as well for the midwest territory of the United States bordering on the Great Lakes. Considering that the present tendency of Federal regulation of the railroads within the United States is to avoid competition in rates and to permit only competition in service, and, further, that Portland is served by Canadian lines which are not subject to regulation by the United States Government, it would seem as if there were possibilities which no other Atlantic port in the United States possesses, for competition in rates and the securing of business.

It is believed by the people of Maine that the development of this port will stimulate the industrial and agricultural development of the entire State, especially since the State has available a large amount of undeveloped water power which, in these days of high priced coal, may now be economically utilized to provide cheap power for industrial uses.

Administration of this new publicly owned terminal the construction of which is to be begun in the Fall of 1921, is in the hands of a board known as the Directors of the Port of Portland. The Board is composed of five members, one of whom represents the city and the others represent the four Congressional districts of the State. The people of Maine, and the Directors of the Port of Portland in particular, keenly realize that their obligations do not end with the provision of adequate, modern, water-front facilities, but that when these facilities are provided and before their completion, active steps must be taken to sell the port to their community and to the country at large.

RELATION OF WAREHOUSES TO PORT DEVELOPMENT

By M. A. Long,* M. Am. Soc. C. E.

The speaker will confine his remarks to his twenty years of experience in railroad work and his study of the warehouses of the different railroad companies, with relation to their inland, lake, and terminal ports.

For instance, at Cincinnati, Ohio, there is a railroad warehouse practically ¼ mile long. Why did the railroad company build that warehouse? Because, if it had not, it would have been compelled to enlarge its yards and buy more cars, using those cars for warehouses in which to store merchandise. By building that warehouse the company saved the expense of additional cars and yards, but a better reason for its construction was to attract business to its line which it had not enjoyed before. The merchants along the line of that railroad also store in this warehouse—advertising it as their own, for that matter—and then ship in small quantities from it to their customers along the line, or along other lines in the adjacent territory. The construction of that warehouse saved these manufacturers the necessity of building warehouses of their own in which to store the surplus which, particularly in seasonal business, they usually have.

If this is true at Cincinnati, is it not true of every seaboard port? Anybody going through New Jersey, and seeing the number of cars on the railroad tracks there, cannot but wonder why the railroad companies do not have warehouses in which to store goods, instead of storing them in cars which are expensive and while being used as warehouses do not earn any revenue, but show a loss. Each year, the railroad companies are compelled to buy more cars and build more tracks to keep up with expanding business. The remedy for this, and this applies to every port, is the construction of a series of warehouses. The speaker is quite sure that, if they were built, the manufacturers of this country would use them and so would foreign merchants.

If a firm in any foreign country, doing business in America, had a surplus of goods, knowing that it could dispose of them to advantage in America, and knowing also that there were warehouse accommodations at one of the terminal ports, it would be to the advantage of that firm to ship them, using the warehouse company as its agent for local distribution; or, if its business was seasonal, ship to the warehouse to hold until it had proper opportunity to market its product. There will be a great deal of business of that character as soon as warehouses are built. Of course, there is a reason for all things, and the demurrage rates charged, or rather not charged, are the factors governing the warehouse situation at the ports. The following indicates some of the comparisons between rates charged on foreign and domestic freight. Let a firm ship a carload of merchandise to an inland port or city, after 48 hours' free time demurrage will have to be paid at the rate of \$2 per day, for 4 days, and \$5 per day thereafter. Suppose that same carload was being shipped to London: if it is shipped to New York on through bill of lading, 15 days' free time are allowed; if it is shipped on con-

* Baltimore, Md.

signment 10 days are allowed; for the first 20 days after the free time expires, 1 cent per 100 is charged, and for the next 10 days 3 cents per hundred, and it is understood that a lower rate is now being put into effect. Thus, it is evident that, while a local merchant can well afford to pay for handling and storage warehouse charges, one dealing in foreign business does not have to pay such charges, because it is cheaper to use the railroad car as his warehouse.

One of the previous speakers, Gen. Beach, has stated that it costs more to handle a ton of freight in the New York terminal than it costs to transport it by rail from New York City to Pittsburgh, Pa. That is true, and it shows that the railroad companies make their profits from transportation and not from terminal handling. If this statement is correct, the railroad companies, therefore, could well afford to sell their terminals to the cities, providing they were guaranteed no loss of the business they had built up at a great cost; and it would be very hard indeed to estimate the relative value of a strategic location in a port, as compared to one not so favorably located.

In regard to railroad piers, the speaker has found from experience that they are seldom filled. In fact, in a publicly owned or operated port, ship schedules can be arranged so that those piers will be working to capacity. The speaker has known a time when the railroad piers would not average one vessel per week. That does not pay, and it is necessary to haul a great many tons over the railroad to make up for this terminal loss.

The Baltimore and Ohio Railroad Company contemplated, and made plans for, a pier and warehouse development at Tompkinsville, N. Y. It was not built, however, at the time it was being considered, because the financial market was not favorable. The Company then learned that the city might take the site and one thing and another led to the postponement of the development, and now the city has taken the site. This is only one case, but, in general, it shows why the railroads have not built modern terminals and warehouses at the ports to provide for future business.

Recently, a railroad man said to the speaker, "We have to build more terminals; it is costing us $1 000 on per diem charges". In other words, the cars from various sources are shipped to this road, because this company has arrangements with other roads not reaching the seaport, to handle their cars, and so many carloads were held on consignment in its yards that the per diem rate which had to be paid to the other roads amounted to $1 000.

If the cities or the public owned the terminals, they would be adequate, the railroads would not have the excess terminal charge, and, with adequate terminal warehouses, they would not have to provide so many cars.

In going through his files, the speaker found a reference to the comparative cost of building cars as against building warehouses. For example, take the Locust Point Yard of the Baltimore and Ohio Railroad Company, which has a capacity of 2 500 cars. There has not been a day in the speaker's experience with that railroad, in which that yard was not, as railroad men term it, "chock-a-block". For $1 600 000 a warehouse, including aisleways, mechanical equipment, etc., could be built to house the cubic contents of those 2 500 cars, so that it would work in with the yard and piers. The cars and land are worth $5 000 000.

If each of ten railroads was to build two of these warehouses, it would be equivalent to building 50 000 cars, as it would release that many cars now tied up in yards due to lack of storage room. Without property or tracks, the cars would cost approximately $100 000 000, while the warehouses could be built for $32 000 000.

The railroad companies are discriminating against the domestic as well as against the export shipper, and have done so for years past, in that they allow their cars to be used as warehouses, and that is the reason the warehouses are not built. It is easier for a shipper to ship on a through bill of lading and get that 15 days' free time, with the hope that a vessel will be available and that satisfactory shipping rates can be arranged before it expires, than to pay handling charges in the warehouse. The value per car and space in a yard is calculated to be $1 500. Interest, maintenance, and depreciation charges on the car and tracks are estimated at $172, while if the same carload is placed in a warehouse on the same basis of interest, maintenance, and depreciation charges, it will cost only $61.

Assuming that the average load per car is 25 tons, the annual revenue, on a storage basis, is $115, or a loss of $57 per car per year, if merchandise is stored in the car, and a profit of $54 per carload, if it is placed in the warehouse.

Since the Locust Point Yard is reasonably full of cars at all times, the speaker has used the yard capacity on a yearly basis for estimating. However, all material would not be stored a full year, and after deducting the handling cost, the revenue would be nearer $45, or $112 500 net profit, for the warehouse, against a loss of $142 000 per year, if stored in cars. These figures are general, but they are relative. The earning value of the car in service is not taken into consideration; the comparison is based on its use as a storage warehouse.

The value of the increased capacity and the removal of the necessity on the part of the railroad companies of buying high priced property in order to increase the capacity of their yards, with the likelihood of adding to congestion, are items which count for more than the actual earnings figured on a tonnage basis; and by keeping the cars more actively in use, the miles per car per day will be materially increased. This, of course, would mean greatly increased revenue, and, after all, the public pays the bill, and if the railroad companies could be relieved of excessive terminal charges, they would be in better shape to accept a reduction in freight rates, in which the public is so much interested at this time.

Take New York City as a concrete example, and assume that it will have its usual normal growth for the next ten years. Can any one imagine what it will mean to enlarge the yards and the cost of additional cars necessary to take care of the additional business? Now is the time to start building warehouses in order to prevent this necessary and expensive expansion.

Relative to the matter of warehouses, about three months ago, some interested parties had an opportunity to get a large manufacturing concern to locate at Baltimore, Md. It was practically all arranged, when, in figuring its operating cost to the *n*th power, the company found that Baltimore did not have proper warehouses for certain vessels, or rather certain ship lines, which were handling the raw materials which it wanted from Europe. This

meant that it would be necessary to ship those raw materials from New York to Baltimore to be manufactured, and have them shipped back again for distribution. Therefore, Baltimore lost that enterprise, and New York got it, because the latter city had a greater percentage of warehouses of that character, and also because it had the shipping lines carrying the raw material. This example is a point in favor of the better terminal, and is the strongest argument that can be brought home to engineers on this subject. There should be adequate terminal warehouses built at every seaport.

FUNCTION OF PORT TERMINALS AS CLEARING AGENCIES

By J. Rowland Bibbins,* Esq.

The United States Chamber of Commerce is deeply interested in, and is organizing to study more intensively and on a National scale, the transportation problems of the country as a whole. The speaker, therefore, will confine himself entirely to one of the broader aspects, as he conceives them.

Four distinct aspects of the subject have been outlined for discussion: First, the technical design, which relates to adequacy, capacity, and efficiency in technical problems, and which any experienced engineer can work out, given the problem of equalizing the capacity of main lines, terminals, and ships, such as will be necessary to handle the business of a port. The second aspect is that of administration, covering execution policy, service charges, co-operation with other transport agencies, development plans, etc. The third aspect is the merchant organization, a term happily devised by Dr. McElwee, covering such functions as banking, warehousing, forwarding agencies, etc., all relating to the merchant business of the port. Fourth, and most important, is the aspect of the port terminal as a gateway—the gateway of the interior. It is the funnel mouth, the clearing house for the nation's business with foreign lands, too often complicated by local business with which it has no relation. The gateway controls the production and the economic development of the interior in a manner often lost sight of in discussions of technicalities.

Commerce is interested in developing economical gateways and keeping them open and unobstructed. Commerce regards transportation as a unit element in the cost of doing business, that is, of distribution. Commerce pays the entire transport bill and does not single out any particular element, such as the port terminal charge, on which to concentrate its demands, unless conditions arise which retard, obstruct, and render the transaction of business unnecessarily costly or hazardous. The thought which the speaker wishes to convey is that in this discussion the total cost of transportation must be kept in mind, from the origin to the final destination of shipments. This may involve five or ten re-handlings, or even more for small lots, namely, from the producing interior, through the collecting agencies, the rails, through the terminal agencies and warehousing, the ocean carriers, through the foreign terminals, and distributing agencies; and *vice versa* on imports. This is the complex problem that confronts commerce, and, of course, the port terminal is an important element in that whole problem. After all, the essential criterion of all transportation is based on two things—time and cost.

Back of these various coastal gateways, is a great producing area of 3 000 000 sq. miles (with an additional area of 1 000 000 sq. miles in Canada which is closely associated with that of the United States). It is said that only 5% of the total business of the United States is with foreign countries, but the speaker suspects that 25% of the raw products flow through the ports.

This internal production from farm, mine, and factory has grown up through long years of development and had adjusted itself to the pre-war plan

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of rates, routes, terminals, business organizations, and manufacturing locations (such as Gary, Ind.). The war dislocated the whole economic plan, that is, the relationships between location of producing centers, transportation costs, and distribution costs. Years are required for an industrial system to readjust itself to essentially changed conditions. If this dislocation continues, the future may force industry to gravitate to the seaboard in order to meet foreign competition. The truth of this statement appears in the great demand for a reduction in freight rates from the far interior; the demand does not come from seaboard points. Therefore, unless the gateways—and by this the speaker means not only the water gateways, but the internal rail gateways as well—become so highly organized as to overcome this handicap of long and expensive interior transportation, a complete realignment of industry in the United States may be witnessed.

Commerce gravitates along the line of least resistance; in fact, if one can conceive of commerce and tonnage as water running down hill, one can visualize the Economic Divide—a hypothetical mountain range running diagonally across the country, determining which way that water-borne tonnage should naturally flow to foreign ports, that is, whether it should flow east or south, or to the west. It is not difficult to equalize the water and rail rates from a given producing interior region to a given foreign destination, to locate definitely this Economic Divide.* Temporarily, the Divide may be diverted by sentiment or precedent; that is, if logic and the true cost of service dictate a certain route for internal tonnage to its foreign destination, and it is found that the tonnage actually flows by an entirely different route, this may be the result of historical precedent, regional sentiment, preponderant rail development, or shipping policy. Eventually, however, trafficways for this interior production must find and rest on their economic justification in order to become permanent. Likewise, the gateways must retain their efficiency in order to enjoy permanency and to meet growth; and the gateways are the keys to the situation.

How shall this demand for continued expansion be met? Analyze the general business indices† of typical cities of the United States and it will be found that the general business which concerns transportation doubles in from 8 to 13 years. Cleveland, Ohio, is an example of a progressive community doubling its business in 8 years.

Now, even a modest port plan would require at least 5 years to organize and start *de novo*. It might easily take 10 years or more to complete the major elements, yet the ton-mileage of the United States doubled in 13 years, just prior to the World War, and its tonnage has increased as fast as the fourth power of the population. Here, then, is some measure of the problem of growth which concerns every seaboard municipality—every port terminal—and the widespread lack of appreciation of this rate of growth is responsible to a great extent for the lack of development. In other words, a different con-

* See the speaker's paper, "Economic Lines of Gravitation for Overseas Movement", Am. Assoc. of Port Authorities, 1920; *Engineering News-Record*, 1919.

† Such as imports and exports, railroad tonnage, shipping tonnage, warehousing, bank clearings, post-office receipts, industrial output, auto-registration, electric railway traffic, school attendance, telephone traffic, etc.

ception of port-terminal development is necessary, if Americans are to meet the obligations imposed by an expanding commerce on a National scale.

Let us now examine a typical large port terminal.* This port includes half a dozen competing railroad-terminal services, all interlocking as a result of previous development through 20 or 30 years of intense competition. Some of the terminals must actually be losing money in the effort, but they must keep going to secure their share of the traffic. This port city has grown solidly around the railroad terminals and the water-front district, enclosing them and restricting the possibility of their proper development.

Of the total car movement, only one-sixth is through land interchange, as compared with more than one-half in Chicago, that is, traffic having no business in the city proper. More than two-thirds of the total car traffic is interchanged between the roads, and two-thirds of this is done on the congested water-front, while only one-fourth of that car movement is direct export and import movement. In other words, city business and marine business is hopelessly interlocked in the attempt to carry on both in the same location. This emphasizes an important premise—that the water-front is not the place for city business, nor is it the place for rail clearing or storage operations.

Now, the peak load of this port is quite severe. The terminal handles 25% more cars loaded and empty during the peak load than during the normal month. It handles 50% more cars received immediately after harvest time, and, at times, the grain receipts are several times larger than the average for the month. The result is that cars pile up in the terminals by the thousands. The seasonal excess (over minimum) may be as much as 15% of the total car receipts, and months are required to eliminate this excess car holding.

This emphasizes the fact that the clearing capacity of terminals depends much on their mobile reservoir capacity—the "liquid assets" of the operating man. That point has indeed been emphasized in this discussion, but it means capacity of warehouses as well as of railroads. The railroads should not be required to provide all the warehouse capacity—a bad practice which greatly decreases the effective utility of railroad facilities.

Railroading is a continuous function; shipping is an intermittent one. The difference in the turn-around time in boats and cars must be met by reservoir capacity. Who is going to provide that capacity? That is one of the great problems of the port plan. If it is not provided, every harvest time will result in a vast car movement to the seaboard, the clogging of terminals, and a serious shortage of merchandise cars in the interior. The coal problem is similar. One 10 000-ton cargo would require 3 miles of cars standing, 2 000 or 3 000 truck loads, or 8 miles of trucks in line. Imagine the activity required in a port terminal designed to handle 20 cargo ships at a time, with an average of 5 days turn-around (which should be possible instead of 10, 15, or 20 days' turning time). What is needed at present is more movement, not more cars or ships. It is the cost of idleness that eats up the profits.

* Based on a detailed technical survey of origin destination.

Summarizing this very brief discussion, commerce is interested in the following:

First.—The clearing capacity of the gateways during heavy traffic seasons. This is analogous to the principle of designing a power plant to carry a peak load.

Second.—The over-all cost of transportation from the producer to the consumer, including the important terminal cost. It is the total cost from moving train to moving ship which controls.

Third.—The time consumed in transit, avoiding expensive and embarrassing delays.

Fourth.—Economic routes overseas which provide least time and least cost.

Fifth.—The provision for continuous development to meet future growth of traffic. In this respect, the newer ports, the younger ports, have a great advantage, because they are not handicapped by expensive precedent.

Sixth.—More consideration of the transport machinery back of the bulkhead, that is, rail, belt lines, and motor transport auxiliaries, are required in the port, as well as the water-side facilities and equipment, likewise so important.

Seventh.—The separation of city and water-side freight transport facilities is necessary to permit each to function without interference. This should be a major feature of the city plan.

Eighth.—Acceleration of carrier equipment. This may determine the success or failure of a port.

Permanency or supremacy cannot be guaranteed to any gateway, for, ultimately, commerce will adjust itself and select its own trade routes and outlets, based on true economic stability.

LACK OF CO-ORDINATION IN DESIGN OF AMERICAN PORTS

By John Meigs,* M. Am. Soc. C. E.

Recently, a conspicuous official of one of the major ports of the United States, in an unfortunate moment of mental aberration, charged the Engineering Profession with responsibility for the lamentable fact that "this country to-day is secondary in importance to Great Britain and other European nations as a maritime power."

This ingenuous and surprising allegation was followed by the further count in the damning indictment that these hyper-technical malefactors—that is, engineers—or, as he neatly expressed it, "impractical and technical men," were animated by motives "sometimes prompted by selfishness, frequently by favoritism, and often by inexperience."

These initial strictures were then amplified by further charges of high professional crimes and misdemeanors on the part of various specific members of this Society, who had the temerity to design harbor structures not in accordance with his own secretarial views—this having been his vocation prior to his present incumbency of a responsible executive post.

If this highly placed official had additionally charged the impractical theorists of this Profession with the responsibility for the World War, for the present financial depression, and the unusual prevalence of mosquitoes on the Jersey coast, the sweeping indictment would have been complete, his animadversions would have been quite as just and logical, and no doubt it would have relieved his mind of the profound depression in which it was then apparently inextricably bogged.

Of course, such unmitigated absurdities as these would warrant no attention, save that they came from an incumbent of a prominent municipal-political office, and that they indirectly "point a moral and adorn a tale."

If, however, engineers reject this kindly meant solution of the problem of "what ails our American ports?", in fairness they presumably must suggest a better one. The uncontrovertible fact is, not that engineers have by their iniquitous dominance been responsible for the comparative backwardness of American port cities, but, on the contrary, that they have had far too little to do with their development, having been in most instances, until very recent years, notable in important port councils mainly by their absence.

It is true that the services of engineers have long been utilized in port design along the more technical lines of harbor planning and in the detailing of various port structures, but, in the larger aspects of the art, the general design and co-ordination of port layouts, they, unfortunately, have not been able to make their influence felt as they should.

Too frequent changes of political administration in the greater number of the ports have made impossible the adoption and carrying out of logical and comprehensive plans, however good the intentions of their proponents may have been, have rendered futile the well meant efforts of engineer subordinates to institute well considered programmes of development, and have resulted

* Philadelphia, Pa.

in piecemeal and inconsistent efforts at isolated betterments. Each successive, short-lived administration has entered into office as a rule suspicious of the plans and procedure of its predecessor, anxious to make an ornate showing for itself, and regardless of the troubles left by it in the hands of its successor.

To illustrate this precession of American port luminaries, the speaker may mention the fact that nearly ten years ago, when the American Association of Port Authorities was first organized, as a humble member thereof he became acquainted with practically all its membership, including the representatives of the ports of the United States and Canada. During this decade, the speaker has seen in the case of the ports of the United States at least three changes of administrative officers and, in some instances, four or five; while in the case of Canadian ports they have remained for the most part in the hands of the same executives for this entire period—men trained, salted, and seasoned by long and valuable practical experience. Not one of the municipal port officials of the United States then in office, is now connected with port administration.

This applies, it is true, mainly to the chief executives of these various departments, but it affects also the working personnel to a considerable extent and makes impossible the proper carrying out of consecutive programmes of improvement.

The better balanced developments exhibited to Americans by European and by some Canadian cities—notably, Montreal—and the superior economies of operation possible thereby, are due in large measure to their rational conception of the proper function of a port administration as a genuine business organization rather than a political or social diversion.

When referring to comprehensive port developments, the speaker does not necessarily mean the somewhat flamboyant and spread-eagle projects designed to provide for the hypothetical needs of unborn generations of men and unbuilt navies of ships with which the port fathers have amused themselves in so many recent instances, admirable in themselves though these plans have often been. Rather, he more especially wishes to emphasize the necessity for proper balance in port layouts, be they large or small in size, and an economic co-ordination of the various parts and functions of the port.

Provisions for the receipt and transfer of freight to and from land and marine carriers are essential of course, but of equal importance must be considered the provision of facilities for its warehousing and merchandising. The complete port consists not only of its channels, anchorages, piers, quays, and transit sheds, but also of proper rail and road transportation facilities, and, not least, ample provisions, both outdoors and under cover, for the short and long-time assembly of cargo and the protracted storage of it in warehouses for eventual reshipment, for resale at favorable opportunities, for repacking, assembling, manufacturing, or other preparation of it for the market.

Have American municipalities devoted the proper study to this important subject as a scientific problem, or are the ports being permitted just to grow, "Topsylike", along the lines of least resistance? To any one familiar with the

trend of port development in the United States, it is not necessary to answer this question. He will know only too well that we are merely drifting.

Much money is being spent in the improvement of the harbors—tremendous expenditures in the aggregate are being made—but how much of this spending is wise and how much of it unwise, or even profligate, is a serious question. When two ports of approximately equal commercial movement are found, one with nearly twice the wharf space of the other and still feverishly engaged in spending on more, and more, and more wharves, it looks as if satisfactory explanations of the "whyness" of this spending would interest the citizen taxpayers who are providing the funds for the possibly misguided expenditures.

When port officials devote their energies solely to constructing spectacular new piers, and entirely neglect the equally important railroad and warehousing facilities to back them up efficiently, and the proper improved mechanical equipment to facilitate the despatch of cargo, it is time for a halt until the entire system of port administration can be placed on a genuine business basis.

Permanent boards, composed of business executives, operating men, and trained engineers, should be placed in charge of these matters, paid decent compensations, and permitted and required to stay on the job. Then and then only—when the ports are administered as business enterprises and not as political opportunities—can Americans hope to compete in operating efficiency with well organized overseas competitors.

No such reckless and extravagant use of water frontage is observable anywhere in the world as in the harbors of the United States, which, provided by Nature with facilities almost ideal as compared with the great ports of northern Europe, have been developed in truly frontier fashion, sprawled out over many times the length and area of other world ports of corresponding commercial importance, and operated with consequent inefficiency.

Per foot of steamer berthing space, American ports show an annual unloading capacity of from one-third to as little as one-tenth that of well planned and properly administered foreign harbors. When one considers what this means in decreased costs of every kind—both in connection with construction and operation—the greater administrative facility, and the more efficient management in every particular, the facts in the case demand most serious attention.

As a typical American example, the municipality of Philadelphia, Pa., has about 15 000 ft. of highly improved, deep-water, wharf frontage or berthing space publicly owned, 65 000 ft. of additional deep-water berths under private control, and, approximately, 50 000 ft. of additional berthing space suitable for coastwise and river-trade purposes, a total of nearly 130 000 ft. of improved wharf frontage. Not a square foot of warehouse space in the port is municipally controlled, and, compared with foreign ports, there is relatively little under private ownership. The warehousing is done largely on the pier sheds, where it was never intended to be—and should not be—permitted; and no effort whatever is made to utilize the port equipment intensively.

Abroad, the port of Manchester, England, handling but slightly less foreign business than Philadelphia, possesses only 34 000 lin. ft. of berthing space, less than one-half of the deep-water berthage in Philadelphia and about one-quarter of the latter's total berthage; but, on the other hand, Manchester has warehousing accommodations amounting to approximately 3 200 000 sq. ft. as against Philadelphia's inadequate quota of 1 200 000 sq. ft. In addition, the Manchester docks are equipped with most elaborate mechanical handling equipment designed to facilitate cargo loading and unloading and to make every linear foot of frontage of maximum cargo capacity.

A little nearer home, in Canada, the Port of Montreal manages somehow to handle an annual foreign tonnage of nearly the size and importance of Philadelphia's deep-water berthing accommodations—and accomplishes this in an open navigation season of less than 8 months, or only two-thirds of Philadelphia's working year.

The case of Philadelphia is not mentioned as unique. It is merely typical of current American practice along these lines, and nearly all the other ports of the United States are justly subject to the same criticism.

Are these inconsistencies of performance to be accounted for by any radical differences in classes of cargo handled, or the industrial surroundings of these several ports? Most probably they are not; and, also, most certainly the current practice on one side of the line or the other is wrong. Are the ports of Manchester and Montreal handling more tonnage per linear foot of available berthing space than they properly should and, at the same time, show a decent and kindly regard for American sensibilities; or, possibly, has Philadelphia and her sister ports more piers and berths than they have any legitimate need for, and which they have not yet learned to use properly?

If, however, the municipal authorities have shown lack of forethought in the study of these matters, private terminal managements cannot be accused of like carelessness, as some excellent examples of well balanced design are exhibited in their plants.

In the Bush Terminal, in Brooklyn, N. Y., for instance, with a total berthing space of approximately 16 000 lin. ft. and a pier area of more than 1 000 000 sq. ft., the warehouse area is in excess of 2 000 000 sq. ft. This terminal presents an exceedingly interesting study, in that although it is by no means a model from a construction point of view, it may be assumed to be an almost ideal layout from the standpoint of the balancing and proportioning of its various facilities in their relation to each other and to the general plan.

The present layout being the result of a gradual growth of several decades, from an insignificant beginning, its various piers, warehouses, railroad, and other facilities have been constructed one unit at a time during successive years in response to the actual demands of the business of the terminal, and, in its present state of completion, it may be fairly considered to be planned almost exactly to meet the accurately determined needs of this particular location. This great commercial terminal is commended to city fathers for their careful study.

Decided progress was made during the World War in scientific designing in the case of some, but not all, of the Government terminals constructed for

the shipment of munitions, and these steps in advance might well be emulated by municipal port planners.

In the great South Brooklyn Army Supply Base, a most radical departure in terminal proportioning was made when, in order to balance 7 800 lin. ft. of berthing space and 585 000 sq. ft. of pier area, there was provided more than 4 500 000 sq. ft. of warehouse floor area. This proportion of warehouse space to pier area of more than 7 to 1 may be far in excess of the correct balancing of these facilities in the average commercial terminal. This, however, is a preferable fault to the contrary one of insufficient warehouse space and a superfluity of piers.

It is poor business to build piers at from $5 to $10 per sq. ft. of deck area, and permit them to be used as storage warehouses, while the latter class of structures can be provided at from one-quarter to one-half the cost of pier area.

When comparatively cheap warehouses and inexpensive mechanical handling equipment will enable the expeditious clearing of the decks of expensive piers and permit of doubling, or trebling, the number of vessels capable of being accommodated at their berths, there would seem to be considerable virtue in a policy of port development calling for liberal warehousing provisions and the installation of ample cargo-handling machinery.

Is it not time that American municipal port executives ceased specializing in piers, and piers alone, and commenced an active campaign of providing other port facilities in proper relationship to their wharves?

Most of the seaboard cities of the United States need improved intra-harbor rail facilities, effective belt-line railroads, up-to-date mechanical equipment, and extensive increases in warehousing space incomparably more than a far-flung series of individual piers practically unrelated to each other and the general port plan.

It has been argued that railroad and warehouse installations are not proper objects for the expenditure of public moneys, that it is all right to build piers, but all wrong to provide the co-ordinate facilities which will make the piers of maximum use to the community. This is the most egregious stupidity.

Who can say where the proper functions of the municipality stop and those of private capital begin? The propriety of such expenditures can be determined only by the extent and urgency of the existing needs of the community, by the exigencies of each actual case.

The speaker is no advocate of undue paternalism in government, and he thoroughly believes that private capital should be encouraged and assisted in all legitimate lines of investment. Private management is nearly always superior to that by Governments—be they municipal, State, or Federal—in point of economy, efficiency, and almost every vital factor of operation. If private money will provide all the port facilities, docks, railroads, warehouses, and what not, let it do it by all means—under proper public supervision of the general plan of these expenditures—but whenever private capital fails to take advantage of its opportunities to be of public service, then the public itself must take hold and provide rationally for its own needs.

M. A. Long, M. Am. Soc. C. E., in his discussion, has already given some illuminating data on the subject of warehousing principles, and the figures previously quoted are merely random examples selected to suggest the possibilities in this line of investigation. The subject is certainly worthy of more careful study than has yet been given it, and the results in the way of improved port economics to be reasonably expected from a careful and conscientious investigation of the entire subject, and a practical application of the vital facts thus discovered, would amply justify its costs.

Let the municipal authorities wake up; let them show a little ordinary business sense; let them give their engineers and shipping experts a chance to work out their port problems along scientific lines—that is, in accordance with the dictates of sublimated common sense—and the United States will no longer remain as the before-quoted distinguished critic has stated, of "secondary importance to Great Britain and other European countries in maritime trade", at least, in so far as the proper economical and expeditious handling of maritime commerce at the water-gates is concerned.

A BRIEF COMPARISON OF AMERICAN AND FOREIGN SEAPORTS

By W. Watters Pagon,* M. Am. Soc. C. E.

To persons content with superficial observation, the most striking fact about the two groups of ports—European and American—is their marked dissimilarity. Yet when a more intimate study is made, one finds, of course, that much the same factors are operative, and the differences that have developed on the opposite shores of the Atlantic are no more marked in their real essence than the differences in manners and language between the British Isles and America.

The fundamental difference is largely topographical. The speaker cannot recall an American seaport of consequence where he was not asked the question, "Do you not think that this is a wonderful natural harbor?" and, doubtless, it was, for most American seaports are natural harbors. A port, however, is more than a harbor, and depends for existence and prosperity on commercial and political geography and on the business acumen of its traders. Because of this many of the European ports are located on rivers, far from the coast, and their harbors have largely been dug, piecemeal, from the land as additional space was needed. It would be quite accurate to say that European ports have natural wharves and artificial basins, whereas American ports have artificial wharves and natural basins. The obvious consequence of this—somewhat accentuated in America by the individualism of the pioneer and the temporary character of his construction—has been to foster the building of docks in Europe and of piers in America.

On first thought this distinction is all important, yet the speaker cannot but feel that it is of little importance. A quay wharf is nothing more nor less than half a pier, and whether the wharf is a masonry bulkhead retaining an earth-fill, or a piled construction, can only affect the economy of doing business, through its influence on capital and maintenance charges. In fact, though Liverpool is typically European in its materials of construction, it is quite similar to American ports in its layout, and other foreign ports have layouts containing piers—even piers of American pile construction.

Just why tideless docks have been used so extensively in all ports of Europe in recent years is not entirely clear. In Antwerp, for instance, the earlier construction was on the principle of the marginal wharf, very much like that at New Orleans, La. In London, there is a theory that the earlier basins were built quite largely because they provided a water area that could be fenced in to protect shipping from pilferage; which theory is supported by the inscription on the West India Docks, dated 1802, namely, "* * * this Range of Buildings Constructed together with the Adjacent Docks, * * * for the Distinct Purpose of Complete Security and Ample Accommodation (Hitherto not Afforded) to the Shipping and Produce of the West Indies at this Port * * *". Hamburg and Copenhagen have used the idea of docks, without gates, because it lends itself to the requirements of their "free

* Baltimore, Md

ports". Rotterdam has real need for large basins—even if no quays were provided—because a large volume of its business consists in transshipment from Rhine and canal barges directly to vessels that are moored to pile dolphins and buoys in the docks. Yet, of course, there are many advantages to be gained by tideless docks in such a port as London, where the tidal range is 25 ft. and the tidal currents are strong.

Therefore, it cannot be said that the common American type is better in general than the European type, or that it should always be followed in the United States. The better principle to lay down is that both types have their advantages, and American ports should develop their newer port works along either line, according as the local requirements favor the one or the other. To follow the American plan exclusively is to blind oneself to the possible economies of the other, a fact which has been appreciated by some of the port authorities, and certainly it is contrary to the ideal of the terminal engineer, which is so to build as to gain for the owner the maximum ultimate economy.

For the purpose, therefore, of understanding the other factors that influence the type of development, let us set aside the factor of tidal range—a fundamental one to be sure, yet one that is purely local to every port.

Next, in order of discussion, is then the matter of relative permanence of construction. On this subject many elements have a bearing, for example, relative age of ports, tides and other physical elements, seat of ownership, density of business, etc.

Relative age of ports will be discussed more fully subsequently, but those foreign ports the age of which is so great that their business activities are well established, can well afford more permanent structures than the recent ports in this country. Yet, this consideration did not cause Manchester to build cheaply, although its port is an infant, even when compared with the Atlantic ports of the United States and when Antwerp was opened to trade again by Napoleon, the structures built were of permanent type. Railroad-owned Southampton is also an exception; so probably this reason does not generally obtain.

Such influences as tides may have a considerable bearing on construction where structures are in the tidal range, but they would have no bearing within the docks where the water level is almost constant. In the United States the range of tide is not a determining factor, because the American type of piers may be found at Baltimore, Md., with a tidal range of 1 ft., and at Boston, Mass., with a range of 13 ft.

The questions of seat of ownership and density of traffic are closely related. Of course, the tendency in all countries is to build public works of permanent materials, and this may have considerable bearing in those ports where city or State or other public authority controls the works. Manchester and Southampton, however, are privately owned, and here permanent construction is also found; and the privately operated "free ports" are of similar nature. Even in ports which follow the leasing system, where the lessee's wishes govern largely, there is no marked difference. Therefore, it is probable that the mere fact of ownership has no direct bearing, to the exclusion of other factors.

On the other hand, delegation of ownership or control of a preponderant portion of a port to a single body—whether public, semi-public, or private—is conducive to concentration of the business over the minimum of space. Density of traffic also goes hand in hand with permanence of construction. Thus, a line of "jitneys" may amply provide for light, suburban, street traffic, but subways are the only solution for heavy metropolitan service, and heavy capital expenditure may be economically justified under the latter conditions.

This is, the speaker believes, the most important factor, and a little detailed study will emphasize the point. It has been stated that certain European ports handle ten times as great a tonnage per linear foot of quay as certain American ports. Assuming that cost of construction of berthing space alone in the two countries is in the ratio of 3 to 1, then it follows that those foreign ports are doing an amount of business certainly three times as great per dollar of capital investment. If only this much is true, it certainly does not speak well for the American extensive system when compared with their intensive system. There are also, however, many other correlated gains which follow from intensive development. There is a further saving in terminal investment and operating costs by reduction in railway trackage and facilities; a similar saving in warehouses and transit sheds; a saving through the changed conditions which justify labor-saving machinery by permitting its constant use; a saving in truck mileage to outlying terminals; a saving of water-front mileage and, therefore, a reduction in value, thus permitting more widespread use of the shores for industrial plants the materials of which are water-borne.

By comparison, the most potent element of weakness in the ports of the United States is the decentralization of a pioneer country. Such a condition as that found in one American port, where only 1% of the harbor frontage is owned by the public, would be impossible in Europe, where the ports are highly centralized, for administration as well as for physical layout. London has its Port Authority controlling nearly all the important quayage. In Copenhagen, 58% of the quayage is controlled by the Harbor Authority, and 70% if city and State are included. Liverpool and Manchester are almost entirely in the control of the local authority, and the ports of the Low Countries are similarly held.

New Orleans, La., and San Francisco, Cal., are on a parity with these ports, and they are indicative of the success of the idea in America. In San Francisco, the Authority has consistently striven to increase the density of traffic by yearly comparisons of the tonnage handled per square foot of pier. The space allocated to a shipper is contingent on the intensity of use shown previously. The Authority at New Orleans reserves the right to place vessels at wharves which are not in use, even though they are allocated on yearly leases.

In marked contrast with the natural facilities offered by American harbors are some of the development difficulties to be observed in European harbors. The extreme range of tide has been mentioned, with its attendant system of locks and dikes. Contrasting with this is the situation at Copenhagen where the tidal range is slight, but where the tidal currents are so strong that a dike 7 300 ft. long has been built across the Sound, with locks for vessels

and sluice-gates to control the currents. The newer portion of the harbor has been reclaimed from the water and enclosed by about a mile of breakwater. Amsterdam for more than a century had difficulty with the depth of water through the Zuider Zee, and has built two canals to the North Sea—one of which was not successful—to maintain its position. The later one is second only to the Panama Canal in section, and was built at a cost of about $25 000 000. Rotterdam, located more directly at the mouths of the Rhine, has had to face the same problem of maintaining its channel depth to the sea, and has expended only slightly less than its neighbor. Probably the most courageous of all is Manchester, which, aspiring to become a seaport, spent nearly $90 000 000 on a ship canal, and now is almost on a par with its former port—Liverpool. In some ports, the State has assisted the city, but the greater the proportion expended for aids to navigation the less there was available for quays, sheds, warehouses, etc., and, therefore, the greater the need for increased density of traffic.

One weighty difference between the continents lies in the fact that American tonnage is predominantly for export, whereas, in many European ports, it is for import or for transshipment. This is notably true of the grain trade. However, Amsterdam and London, for instance, have extensive warehouse facilities for goods awaiting re-export, which largely overshadow the other port facilities. Much of the wealth of these European countries has been derived from such storage of goods awaiting a favorable market, in addition to the profits accruing to the local ship-owners from transport of the cargo.

The "free port" is peculiar to Northern Europe, where it has been fruitful. Developed to meet a local situation, it has spread to foreign countries. From the traffic standpoint, it is much the same as an interurban railway terminal, where the dense mass of passengers from subway or elevated railway are distributed to the radiating lines which end there, or *vice versa*. From the economic standpoint it is a wedge, a re-entrant salient, in the customs frontier of a country, which extends the commercial freedom of the seas to include a safe harbor where goods may be safely transferred, blended, manufactured, or stored, awaiting continuance of the journey to other countries. It is a "bay" connected with the free ocean and enclosed on three sides by the customs, whereas a bonded warehouse is an economic "island" surrounded on all sides by customs.

Because of its nature, a free port is, in general, owned by the State or Harbor Authority and leased to a private lessee. When the Government of the United States adopts the principle of the free port, which now seems entirely possible, it will have at New York, Baltimore, Norfolk, and other ports, a series of Ordnance General Supply Depots with berthage, railways, storage, fencing, and all other adjuncts of a free port at hand ready for lease. At least one of these ports can easily be developed to equal the present size of the free port at Copenhagen. For equal success, however, there must be aggressive commercial activity to provide the business.

In spite of superficial differences, therefore, there are few factors of importance which are not operative on both continents. Of course, most Euro-

pean ports have much the advantage in point of age, for Tacitus writing in A. D. 61 mentions London as having "a number of merchants and trading vessels", and several Continental ports antedate the Norman conquest. Yet the rise and fall of ports has been rapid, and whereas Antwerp had become one of the foremost ports of the world in the years from 1500 to 1560, she had declined to almost nothing at the end of the century, owing to civil wars and political dominance of other States. Thus, age can create only an historical background, which may facilitate the inauguration of some new venture, but lack of age has been no deterrent to new ports where the local spirit was aggressive and conditions were ripe.

Whether conditions are ripe for American seaports no one can predict. It hinges quite largely on the continuance of the merchant marine. No European city has become great through the activities of its non-citizens, but through those of its own people, who owned their ships and brought the goods to storage under their own house roof or warehouse. The profits from vessel and cargo went to support the Rubens, the Michael Angelos, the engineers, and other professional men, as they will in the United States—if the merchant marine is made to flourish—to those ports the citizens of which own and operate their vessels and lines.

SOME OBSERVATIONS ON PORT FINANCES

By Edwin J. Clapp,* Esq.

The speaker is not going to discuss the subject of an ideal method of collecting port revenues, nor consider, except briefly, the various expedients now practiced in raising enough money to operate port facilities. He is going to discuss, primarily, the practical financial difficulties that confront public port construction work in the North and South Atlantic outports, because of the established practice of the railroads in offering free berths to steamships in the foreign trade. All the Atlantic ports of the United States, except New York, are called outports.

Briefly stated, the situation is as follows: Each railroad at its chief port, often at several of them, maintains a complete set of oversea terminals, consisting of water-front yard with switching equipment, open and covered piers or wharves, grain elevators, cranes, and other machinery for handling bulk freight. Generally speaking, it maintains and operates this ocean terminal as part of the railroad system, just like the team tracks and freight houses of its land terminals. Rail rates cover not only the cost of transportation, but the greater cost of maintaining and operating the terminals. No charge is levied on the motor truck which backs up to a freight house to deliver or receive freight. No charge is levied on a steamship that backs up to a pier to deliver or receive freight. The interest, depreciation, maintenance, and operating costs of the pier are covered by the rail revenues earned on goods hauled to and from the ship.

The outports, intent on increasing the volume of traffic flowing through them and the number and frequency of their overseas services, have found marked disadvantages in this system of railroad ownership of pier facilities. The railroad having invested heavily in a terminal, tries to monopolize all competitive traffic that passes over it; that is, all traffic to and from points reached by the railroad pier owner or its connections—and this includes practically all the traffic the steamship carries except local port business. Other railroads are kept from interchanging traffic with the steamers when the pier owner refuses to switch their cars, or levies on these cars such a heavy switching charge that the other roads are discouraged from carrying traffic the revenue of which is thus diminished. In either case, the steamship line tends to find itself confined to the service of the carrying power and the soliciting force of one railroad, instead of all the lines centering at the port.

Likewise, the railroad pier owner generally bars or restricts the use of its pier for the accumulation of local cargo delivered by teams, lighters, or the short-distance motor-truck common carriers. This is business on which the railroad gets no revenue. Such traffic occupies pier space which railroad revenue freight might use. It is good business for the railroad, but bad business for the ports the traffic of which would expand if all piers were freely open by rail, highway, or water to all inland carriers. The railroad, however, handling all of 500 000 tons of freight moved through its ocean terminal, can make more money than by handling one-half of 800 000 tons. At least, so the railroad officials have calculated.

* New York City.

An efficient port is a funnel through which are poured the exports and imports of a wide hinterland. Railroad ownership of ocean terminals has interfered with the freedom of that flow in the outports. The outports see their disadvantage when they compare themselves with publicly owned ports like Montreal, Que., and New Orleans, La., where the berths for steamers are owned by the Dock Board and the facilities are connected with each railroad by a public belt line. Each steamer is thus cut off from exclusive connections with any rail carriers. For a moderate, uniform switching charge, covering only operating cost, the belt line switches cars between any berth and any carrier. Every steamship line has every rail carrier working to create traffic for it, and all on equal terms. In New York, steamers do not dock at the railroad piers, but across the harbor at piers in Manhattan, Brooklyn, or Staten Island. Railroads deliver freight to the steamers by lighterage. The intervening water serves as a belt line to cut all the water carriers from all the land carriers.

An outport sees that public piers are preferable to railroad piers from a traffic standpoint. Moreover, the outport sees the publicly owned ports going ahead with great extensions, such as have been completed or are in process at New York, Montreal, and New Orleans. In general, it can be said that in the provision of new facilities, the outports have stood still; the railroads are building no new piers and have built none for years. This is partly because the export and import business has declined from a major to a minor element in the traffic on the railroads, and the railroads have not the money to build. The insistent demand for money to rehabilitate the roads and equipment of carriers, and to expand local terminals for domestic traffic, will absorb for years all the funds the railroads can raise, and there will be nothing left to spend on ocean terminals.

Thus, not only is it better policy for the city or the State to supply piers for oversea carriers, but it is the only way in which port development can proceed. However, if an outport erects a pier, or a group of piers, it finds it difficult to make the steamships pay anything for their use, because a few blocks away the railroads offer their piers rent free. The railroads recoup themselves by their rail earnings. The city has no direct earnings out of which to pay the cost of carrying and supplying piers free to steamship users. Few cities are financially able to carry this annual loss for the sake of the general advantages which port development brings to commerce and industry. How are the outports to finance their new public piers, meet the cost of their interest, maintenance, depreciation and supervision, or operation? There are four ways in which this money can be raised.

First.—The piers can be supplied free and the annual cost taken out of the city taxpayers. This is not practicable. Indeed, it has become customary for the voters or their representatives to authorize port expenditures only on condition that they are to be self-supporting.

Second.—The annual cost of a pier can be met by a dockage charge, levied against the ship each time it uses the pier and proportioned to the size of the vessel and the length of its stay. This is a main source of revenue at New Orleans, where it is supplemented by a "sheddage" charge levied on the ship

for the use of covered space, and by an annual "preferential assignment" charge per square foot of space paid by the steamship line for a semblance of permanency in its berth. In some cases, these charges against the ship are commuted into an annual rental, as at New York, where the pier is turned over to the steamship line for its exclusive use.

Third.—The annual cost of a pier may be met by the imposition of a "wharfage" charge levied on the goods that pass over it. These revenues may be supplemented by storage charges levied on the goods which do not simply pass over the pier, but remain there for a time. This method of financing piers, developed by the railroads, is practiced at the South Atlantic ports. They have long collected charges according to a "wharfage, storage and handling" tariff, the charges being added to the rail rate on southeastern export and import traffic.

For years these charges have been far too low to meet the cost of maintaining and operating the piers. Since January, 1921, a new tariff has carried charges designed to be adequate. At some ports such wharfage charges are not added to the rail rate, but paid by the railroads out of the rates. This is done at Montreal. Its chief source of revenue is wharfage, most of which is paid by the railroads out of their earnings. Montreal has a very scientific system of raising its revenues. The wharfage charges already described are kept high enough to provide interest on the piers. The berths are then leased to steamship companies at rentals sufficient to carry the annual cost of the sheds. Elevation and storage charges make the grain elevators of the Montreal Harbor Commission self-supporting. Switching charges paid out of rail rates to the harbor belt line are designed to make the belt line carry itself. At Galveston, Tex., a similar method prevails. The ocean terminal facilities are owned by the Galveston Wharf Company. Its revenues come mainly from wharfage on goods allowed by the railroads out of their rates, and dockage on ships, paid by vessels according to their size and duration of stay.

Fourth.—The entire cost of maintaining and even operating piers may be met out of railroad revenues, as is the case with the railroad-owned piers at the outports.

As it is not practicable to propose that publicly owned piers at the outports shall be maintained at the expense of taxpayers, the only way to finance them is by the dockage or rental method and by the wharfage method.

To rent public piers on a self-supporting basis will long be a matter of great difficulty at the outports. The speaker is not unmindful of the experience of Philadelphia, Pa., a railroad port which, ten years ago, bravely "took the bull by the horns", built public piers, and rented them for what they would bring, alongside railroad piers offered free. For many years the rentals were almost imperceptible. To-day, some of the newer berths rent on a basis to carry them, but the net income of the Dock Department, after taking out administration and maintenance expenses, is an insignificant return on the $12 000 000 expended thus far. Philadelphia frankly set out to carry its piers, like its highways, at public expense. No other outport can afford to imitate her.

In the speaker's opinion, it would be a mistake for an outport suddenly to attempt to finance piers on the basis of rentals or dockage collected from ship lines, even if this method were practicable. Everything tends to gravitate to New York. Free berths at the outports have been an inducement to more than one steamship line to establish there. When the railroads were built from the seaboard into the Middle West, the Erie Canal had already concentrated the exportation of grain and grain products at New York. The new railroads terminating elsewhere set out to induce the flow of this traffic through their ports also. They offered the steamships free berths and they also offered larger earnings than obtain at New York—larger steamship earnings because of the "differential" rates (lower than those which then applied to New York), which the railroads terminating at the outports were willing to accept. The outports have fought the abolition of the differentials. They should go slow in abolishing free dockage. Examination of export and import figures proves that the drift to New York is not growing weaker, but stronger. Such new lines as the outports get are generally the result of small beginnings. A free berth is a constant inducement to such experiments.

The only remaining method of financing new public piers at the outports is by means of wharfage levied on the goods. This is the method that should be chosen. It presents no complications at the South Atlantic ports where the rail carriers already levy wharfage and handling and storage charges, in addition to the rail rates. When new public piers are built, the rail rate will set the car on the city pier. Then the city can levy the regular tariff, handling, wharfage, and storage charges. These are designed to be sufficient to carry the old railroad piers which suffer under very heavy maintenance costs. The tariff charges, therefore, should more than carry new modern city piers.

At the North Atlantic outports, financing public piers by wharfage charges will not be so simple, because these charges cannot be added to the rail rate. As explained, the rail rate to a North Atlantic port includes delivery alongside the ship. The railroad, without additional charge, unloads the car and charges no wharfage on its contents. At a city pier, the railroad would deliver and unload the car. If the city tried to charge wharfage on the contents, this charge, added to the rail rate, would throw the cost of shipping *via* this pier "out of line", compared with the adjacent railroad piers. That is, at North Atlantic ports, if wharfage is to be collected at city piers, it must be collected not from the shipper in addition to the rail rate, but from the railroad out of the rail rate. This would be no new practice for the railroads. Already out of their rail rates they pay a wharfage sufficient to cover the overhead and the operating cost of their piers; they pay this wharfage to their own terminal units. Why not pay it to a new independent terminal unit which relieves them of the terminal service? Then the railroad would have the same net revenue for its haul whether the shipment passed over its pier or the city pier; in either case, it would retain the rail rate less a terminal deduction for wharfage. Such rail-rate terminal allowances made to city terminals by the carriers would make city terminals self-supporting. This would make it simple to obtain public funds to any desirable amount.

In other words, this is the situation: In the North Atlantic outports, ocean terminal facilities have been supplied by the railroads the rail earnings of which include a quota to pay for the upkeep of these terminals. Expanding commerce, larger ships, the advance of the engineering art, all require new and improved facilities. The railroads are financially unable to supply them. They can be supplied out of public funds, if the railroads will make to the new terminals the same terminal allowance they would make to their old terminals. Of course, the carriers would be glad to have the outports build piers and make no terminal allowance at all. They make no terminal allowance to the city piers at Philadelphia. On traffic which the railroads carry for movement over these piers, they retain the full rail rate, both that part collected for the haul and also that part collected for the terminal service, which includes supplying a pier. The cost of supplying the pier is thus thrown on the city which shifts part of it to the steamship lines as rentals, but most of it on the taxpayers.

To reduce the suggestion to concrete form: Suppose a North Atlantic outport builds a modern terminal unit, consisting of open and covered piers, a grain elevator with galleries to each steamship berth, a supporting warehouse, an adequate railroad yard, and a belt line cutting the main break-up yard of each rail carrier. Each rail carrier would pay the belt line an adequate switching charge for taking a car and setting it at its berth in the new terminal. This would supplant the service rendered by the railroad's switching engine in moving the car from the break-up yard to its own pier. The railroad car at the public terminal would then be unloaded by the railroad's own men sent there or, better, by the terminal's men, and the railroad would pay a proper handling charge for the service thus performed for it. Finally, the railroad would allow the terminal a per ton wharfage equal to the interest and maintenance cost of its own piers distributed over the tonnage handled.

The railroads themselves would profit from such an arrangement. They would be supplied with new and additional facilities without cost, save as they got traffic to move over those facilities, and, then, at the same cost which they assume in the case of traffic moved over their own piers. At some of the outports, the carriers stand enormous losses by carrying cars of exports under load, because they have no adequate pier space to hold them. The initial wharfage allowance required of the railroads could be reduced in the course of time. The city's overhead on its terminal would be less than that of the railroads; the city could get its money for 6%; the railroads pay 8 per cent. The railroads pay 3% taxes; there would be none on city property. The overhead on the money invested would thus be 5% per annum less than if the railroads themselves built additional terminals. Maintenance at the new piers would be far less than at the old railroad structures.

The wharfage required of the railroads, in the course of time, could be further reduced as new sources of revenue arose for the new terminal. The larger steamship lines seeking it would be charged a "preferential assignment" for regular berths. Some of the lines would require special types of shed or equipment, but such lines could pay interest on the special facilities provided for them, a rental carrying the pier superstructure, as at Montreal. In all

likelihood, the wharfage required of the railroads would be in time only a fraction of what their own piers now cost them in wharfage. It is not impossible that in time the new terminal would become quite self-supporting. It is a plan by which the railroads could gradually work out of the heavy terminal expense they now carry on export and import traffic.

The speaker realizes how sketchy this discussion has been; he knows the difficulties involved. The new terminal should be built and operated only by the highest type of public commission such as have been described by Mr. Cowie and Gen. Beach. The best commission or best authority is one representing the business interests which handle that transshipment which is the port's function. On this Port Authority, both railroads and steamship companies should have adequate representation. Of course, the new terminal units must be built gradually; the railroads should not be asked to help finance terminals which will simply empty those already in existence. There is no use in bedeviling the railroads; but for them there would be no outports; they developed them. The time has come when further development is beyond their financial powers, when ideas of the benefits of competition no longer extend to approval of separate railroad-owned ocean terminals. The speaker believes that the railroads will be found ready for co-operation in the manner herein outlined. If they were not ready, he believes that the Interstate Commerce Act, the Transportation Act, and the Merchant Marine Act, give the Interstate Commerce Commission power to compel such co-operation. The main evils of the present railroad-owned piers can be eradicated by the introduction of reciprocal switching among them, the switching charge to include adequate wharfage for use of one carrier's pier by another.

What is here proposed is that the transition from railroad to public ownership of ocean terminal facilities in the North Atlantic outports proceed without abolition of the present railroad practice of allowing out of rail rates a wharfage sufficient to provide interest and maintenance on piers for oversea carriers. It is proposed that a city commission of port business men construct a single terminal unit, as part of a port plan, and that the rail carriers allow to this new terminal, out of rail rates, the same wharfage they now allow their own terminals. It is asserted that this initial wharfage paid by the railroads can soon be reduced, because of lower overhead and maintenance costs applying to the public terminal and because the terminal will develop revenue from "preferential assignment" leases and outright berth rentals, as well as from wharfage on traffic brought by motor trucks and lighters which will have free access to the new piers.

Neither the outport nor its railroads can afford to see its port facilities stagnate. An attempt has herein been made to present a plan whereby these facilities can be fairly developed, to the advantage of both railroads and the port, without disturbance of existing rate or traffic conditions and with that proper observance of the local situation which must modify any attempt to attain ideal conditions.

IMPROVEMENT AND DEVELOPMENT OF PORTS

By Carroll R. Thompson,* M. Am. Soc. C. E.

In the improvement and development of ports there is no question of more vital importance than the powers that are vested in the body entrusted with the control and administration of port matters. The best laid plans for increasing efficiency will fail in their ultimate purpose unless the port control is such that the various unit facilities can be co-ordinated into a smooth working whole. A single port unit may be designed and built and be capable of operating with the utmost economy and dispatch, but unless the other units are coupled with it to produce the same efficiency, the whole operation of the port will be inefficient. The absolute regulation of each and every element entering into port problems must be controlled by a port body, which, in turn, must be established, organized, and maintained so that it can be administered exclusively as a business proposition.

The work of the engineer is dependent on this control. How can the engineers of a port commission build supporting warehouses to operate in conjunction with pier sheds, unless the commission has the authority to build warehouses? How can the rail facilities in a port be operated to their utmost efficiency unless the belt line is operated on a belt-line principle? What is more discouraging to a port official than to find one of the port's most modern and fully equipped general cargo piers on which a preferential is given to a particular railroad company's freight, or on which one railroad is permitted to run its cars to the exclusion of all others? When such a condition exists there is only one result: The railroad freight that is not permitted to be run on a pier must be rehandled at a distant point and transported to the ship either by lighter or truck. The belt-line principle of operating to allow any railroad carriers to enter on the tracks laid on any pier must actually be in force and be subject to the control of the port authorities, a control which can compel that principle to be enforced.

This is true with every other unit and element connected with the operation of a port. The regulation of privately owned water-front facilities also enters into this question of port control, particularly in some of the older ports. Sometimes attempts are made to utilize tide-water frontage for other than shipping purposes. Unquestionably, the primary interest of any community in its port lies in its shipping development, and, therefore, docks for ships are of the foremost consideration. Water-front structures for private storage purposes or other uses should not be permitted.

Recently, in Philadelphia, Pa., a decision was made denying the plea of a large corporation for a permit to extend a pier to the pier-head line where the dock or water space on each side of the pier was less than 70 ft. and the pier extension only 55 ft. wide. In addition, it was proposed to erect a superstructure covering the entire area of the pier without any provision whatever for an apron. The proposed structure was also of such character as to indicate clearly its intended use as that for storage and manufacturing, or a combina-

* Philadelphia, Pa.

tion of both, rather than for shipping or commercial purposes. The absence of any provision for cleats, bollards, or any other means by which a ship could be made fast, confirmed this belief. Furthermore, the company entered into an agreement with the adjoining property owner by which it agreed that it would not dock any craft of any kind or description along one side of the pier, all of which indicated to the Dock Department that such a structure was in violation of the laws now governing the port. There was involved in this application serious questions—the extension, for instance, of a pier of inadequate width and inadequate water space and the apparent desire on the part of the applicant to secure for manufacturing and storage purposes an area located on the recognized property of the Commonwealth of Pennsylvania.

The Director of the Department of Wharves, Docks, and Ferries, in denying the application, stated that he was alive to the importance of the industry in question to the Port of Philadelphia, and recognized that every facility should be afforded for its further development, and although he recognized the fact that this particular corporation was handicapped for storage room, he was also mindful of the responsibility resting on him as an agent of the State to dispense its bounty and felt that the time was opportune for calling a halt on attempts to secure by license the use of State property for private purposes. In this case, the State property referred to, is the bed of the river between the bulkhead and pierhead lines, on which structures for maritime purposes only can be built.

If sustained by the Courts, this decision will have a far-reaching effect in advancing the development and improvement of the Port of Philadelphia, in that, as pointed out in the opinion, the sanctioning of a non-maritime structure could not be otherwise construed than inimical to the development and best interests of the port.

Attention is called to this decision to show the importance of the control of a port authority over privately owned port facilities and also the fact that the port authority will be less handicapped in its improvement and development programme when it becomes the owner of the riparian rights along the entire shore line.

There are many such problems confronting harbor authorities in their efforts to improve and develop the facilities of a port and very often they are helpless to correct inefficiencies, due to the lack of authority to control all the elements that enter into questions of port government. In one American port, some of the laws regulating port matters were enacted more than a century ago and have not been modified in accordance with the advanced and changed methods of the shipping business.

To reap the full benefits of the improvements and developments that may be made to the harbors, or even properly to carry on successfully the actual construction work of improvement and development, the port authority must have the power that will enable it absolutely to control and regulate each and every element effecting the operation, directly or indirectly, of the port as a unit or a series of units. Its work must be separated absolutely from politics and handled exclusively as a business proposition and it must not be organized and established so that a periodical change of directing heads will

cause a periodical change of policy. Continuity of policy must be preserved. When a comprehensive plan of improvement and development is approved and adopted, the work should be carried on and not fail through lack of authority by the port body. When the port commissioners, or the authorities with whom the control of the port are intrusted, are given powers that will enable them to control or regulate every question concerning the functioning of a port as a whole, the improvement and development of that port will then be much simplified and the port engineer will be much less handicapped in solving his problem of how to produce efficient terminal facilities.

PORT PROBLEMS IN NEW YORK

By John A. Bensel,* Past-President, Am. Soc. C. E.

The subject under discussion has been announced as "National Port Problems". As far as the speaker knows there are no National ports in the United States and, therefore, there are no National port problems, unless problems of immigration or Custom House operation are considered. These are the only National operations in American ports, but as they are not of direct concern to engineers, it does not seem probable that they come under the subject.

On account of these facts, the speaker proposes to discuss some of the problems affecting the Port of New York, as he sees them.

The first question which naturally arises is, what are the problems that affect the Port of New York and thus endeavor to find how they can be solved, if they need solution. At this point, it would appear that one enters into a land of mystery, but the guides are numerous and so are the solutions.

It is quite evident that to many the most important problem in the Port of New York is how to get rid of the water. Many solutions of this problem have been proposed, but probably the one which has occurred to one engineer —of simply filling in one or two of the rivers and a portion of the Bay, all of which appear to be entirely superfluous—is the simplest and easiest to understand, and to him at least the problem is solved.

Other schemes to eliminate the water appear in two quite different forms, one of connecting the mainland of New Jersey with Manhattan Island by means of a gigantic bridge and the other of establishing an indefinite number of small-bore tunnels, built far below the street grade, on Manhattan Island.

The public must be somewhat confused by the solutions proposed, one of which would make a track yard on the most expensive real estate in the world and the other would deposit the freight destined for Manhattan at locations where it is difficult to conceive that it could reach the consignee. In neither of these proposals is an explanation given of the reason for them.

The Port of New York is apparently holding its position eminently well. It does more business than its ancient rivals, London and Liverpool, do in combination, and practically one-half the trade of the country goes through its gates. It is not shown that freight movement would be any cheaper, nor would it by any possibility be any more rapid, and the question naturally arises, "why bother?"

In this connection, nearly thirty-five years ago, certain eminent engineers reported on the problem of how to increase the commerce of the port, and arrived at the novel conclusion that a four-track surface railroad with a track connection to every pier was the solution. Had this joke been perpetrated, the water-front would have been rendered inaccessible and the capacity of the port curtailed.

The great ports of the world owe their importance, not so much to their geographical layout, as to the commercial activity of the back country, and

* New York City.

the Port of New York is no exception to this rule. Had there not been a natural port and harbor, the commercial necessities of the great State of New York would doubtless have produced one, for human ingenuity has known no limit under the pressure of commercial necessity. New York is one of the few great ports located on the sea and, therefore, escapes many of the problems affecting several of the other principal ports of this country and most of the famous ports of Northern Europe. To note some of the problems that New York has escaped, a contemplation of some of the difficulties that have been overcome, is reassuring; Philadelphia is 94 miles from the sea; New Orleans and Montreal are both examples of river development; Seattle and Portland are also examples of port development under adverse conditions; while, in Europe, the situation of the great ports—Liverpool, Manchester, London, Antwerp, Rotterdam, and Hamburg—brings to mind the natural obstacles that have been overcome in their development. All this brings to mind that, perhaps, the problems of the Port of New York, or of any other port, are not really National; they are local issues or problems. This was said once about the tariff, and although it took about twenty-five years to demonstrate it, it was finally generally admitted to be true. It is to be hoped that Americans learn more quickly now.

Most problems mentioned in regard to freight movements in the port, pertain to the stated necessity of placing the steamship and the car in direct contact, a matter that is not at all difficult when desirable. Many localities are available on the water-front of more than 500 miles, and the problem only becomes complicated when an endeavor is made to move freight cars around in congested localities into positions that are clearly foreign to their purpose or design. All the New Jersey portion of the water-front of the port is easily adaptable to such a treatment, if desired. The speaker recalls that when the so-called Chelsea improvements were built, none of the great trans-atlantic lines would allow railroad tracks on the piers, although the freight tracks of the New York Central Railroad were laid on the marginal street directly in front of the piers.

It is certain that quite a new type of engineer has appeared, a type that would aid any scheme provided there were sufficient funds ready for dispersal. Reference is not made to natural differences of opinion which must always develop in professional activities, but this new type only endeavors to silence the naturally curious by vociferous assertions of what they know, given, of course, without figures, and it would appear to be clearly propaganda for some hidden interest or the development of some commercial scheme. It is certainly a problem to obtain an unbiased opinion, although the experts for both terminals and ports were never more numerous. Bred to a certain extent by the conditions of war work, they are found in every problem, self-elected and ready to advocate almost any scheme for the expenditure of public funds.

The older methods of proving one's point of view must be restored if the engineer is to take a professional stand before the public. Many of the so-called problems are invented in order that the self-constituted expert or authority may get "his place in the sun", but the time must come when proper

analysis becomes the vogue. Thus, many so-called problems will disappear and much that is now written may be put away until needed.

The port problems of to-day do not differ materially and may be briefly stated as being concerned to a large extent with the commercial connection between the rail or canal carrier and the transatlantic ship. Commercial needs must dictate in the methods to be used, and no servile imitation of foreign methods will comply. The great systems of many of the foreign ports are not to be reproduced however effective a lot of cargo hoists may look against a sunset sky.

As the speaker sees the great problem of the Port of New York, it concerns the effective use of the water as a means of communication between the railroad terminals on the New Jersey shore and the points of delivery nearest those of consumption on a shore line of nearly 500 miles. For freight, there is nothing so economical as movement over a water lane, since no railroad yard admits of such expeditious and economical movement of freight cars as the slip between two piers, and the solution of many problems concerning freight movement may be best expressed by the term, "use the water". Do not think that it is necessary to go under it or high above it. The oldest means of movement, it still possesses all the advantages that it ever had and also is frictionless. It offers an almost ideal opportunity for movements on its surface.

As an illustration, it might be mentioned that there arrived in the Port of New York, recently, a barge carrying as much grain as could be carried by a full train load of about 70 cars, all within a bulk about 300 ft. in length, 35 ft. beam, and 12 ft. in depth, and this quantity of grain, amounting approximately to 2 300 tons, made the run from Detroit, Mich., to New York City in 8 days. Does not the picture speak louder than any words? Avoid the problems—"use the water".

RELATIONSHIP OF RAIL AND WATER CARRIERS

By William J. Wilgus,* M. Am. Soc. C. E.

In times like the present, when an impoverished world is practicing rigid economy as never before, it is to be expected that trade between nations will flow along paths of least resistance and, hence, through those gateways which, other things being equal, exact the least tribute in tolls and time from the rail and water carriers which meet there for purposes of interchange. This admitted, can it be denied that American engineers owe it to their countrymen to exert every force at their command to point out in what respect the principal ports of the United States are lacking and how they may be bettered? It would seem that the first move in this direction should be the bringing together of the transportation interests on land and sea, with a view to perfecting their related facilities in a manner that will best serve their common good.

The usual pier or quay, after all, is a joint terminal, not only for the ship, but likewise for the railroad and for the motor truck. Unfortunately, too often this common ground is under the exclusive jurisdiction of one of these agencies, with dire results to the others. For instance, a steamship line in the sole possession of a pier will seldom, if ever, provide proper space on it for tracks on which the railroads may place their cars for delivering and receiving freight direct. It may dictate to the railroads, without expense to itself, that the interchange shall be effected through the medium of trucking or of floating equipment with its added cost of breaking bulk and of tortuous water-front and marine operations. This situation which obtains to-day in the greatest American port—New York—was brought about originally by natural conditions which antedated the railroad area, and, later, was perpetuated through the provision in the seaboard rate which requires the railroad to interchange freight at the ship side rather than at the end of the rail haul.

The wastefulness of this process can be best illustrated by comparing it with the confusion, delays, and excessive costs that would reign in a joint railroad terminal where different gauges of the main lines and yard tracks would necessitate the breaking of bulk or transfer of car bodies in transit, within sight of their final destination, a condition too absurd for serious consideration.

The remedy for this manifestly unhappy situation lies in the conversion of port authorities and carriers, both rail and water, to the wisdom, or, rather, strict necessity, of abandoning the stingy pier policy and of building generously proportioned wide piers and quays on which there shall be ample space for transit and storage sheds, motor-truck driveways, and track layouts designed for continuous cargo handling uninterrupted by switching operations. All the great ports of the Old World, and even the principal ones of South America and of Canada, have taken this foresighted course, and in the United States the more progressive ports have rather timorously done likewise. There are, however, many ports, notably New York, where the narrow-pier policy still rules. In planning for the future then, let past faults be remedied and

* New York City.

wide piers substituted for the inefficient and wasteful narrow, trackless ones which have been inherited from the days of the sailing ship and canal-boat.

Another direction in which there is a dire need for improvement is the more extended use of mechanical equipment for reducing the expense of cargo handling and for expediting the release of ships and cars. For example, during the World War, the American Army in France operated in twenty to thirty ports at which in the last month of the war, November, 1918, there were discharged, by a variety of methods, nearly 1 000 000 tons of supplies at an average rate of 449 tons per ship per day, or, approximately, 1.10 tons per lin. ft. of ship berth per day. At one of these ports, American Bassens, there had been completed at that time a partial installation of electric gantry cranes, which despite many handicaps, such as untrained operators, poor lighting facilities, shortage of cars and stevedores, and insufficient "tuning up", made an average record of 717 tons per ship per day, equal to 1.8 tons per lin. ft. of ship berth. This was in marked contrast to the 371 tons per ship, or 0.9 ton per lin. ft. of ship berth, discharged daily at neighboring berths where non-completion of the gantries made necessary the handling of similar cargoes by ship tackle, not only with less speed, but also with greater damage to the supplies and with much more rehandling between ship, car, and shed by reason of the shorter radius of action of the ship tackle.

The effect of the gantry installation on economy of ship time and car time and on man-power is further illustrated by extracts from test observations taken at the same ports, as shown in Table 1.

TABLE 1.

Berth equipment.	Average tons per hook-hour.	Average tons per man-day (10 hours.)	Time of ship in port, in days.
Gantry crane	14.5	6.0	11
Ship tackle	8.4	3.7	16
Superiority of gantry crane over ship tackle.	73%	62%	31%

It will be seen that in this instance of mixed cargoes ranging from Quartermaster supplies to railroad materials, machinery, camions, guns, and fighting tanks, the modern mechanical equipment, even with the handicaps mentioned, was pronouncedly more efficient than the methods in ordinary use.

It is believed that these war experiences, which by the way were borne out by those of the British Army in France, should be applied to peace-time commercial needs. At least, they are worthy of thought in connection with the planning of labor- and time-saving devices at the National gateways.

As bearing on the wisdom of a wide-visioned improvement of American ports, it will no doubt be of interest to cite the outcome of a recent study for a large port extension, which indicated that, although the widening of the piers for the accommodation of suitable track layouts, cranes, transit sheds, and driveways, would increase the construction cost over narrow piers more than 50% and reduce the linear space for berthing ships, the additional cost of the plant as a whole, including not only the piers but also the ships, cars, and

motor trucks, was only 8%, while the reduction in berthing space was more than offset by hastened speed of cargo handling and therefore quickened release of ships and cars. In consequence, it was shown that, with an undiminished annual tonnage capacity, the efficiency of the port would be greatly enhanced and the terminal charges, embracing ship and car demurrage, as well as labor, reduced more than one-third, after making due allowance for the increased fixed charges.

These figures, of course, are not claimed to be applicable to all times and places. They may be said, however, to serve the purpose of directing attention to the outstanding wisdom of carefully analyzing each port problem at which rail and water carriers interchange freight, both as regards adequate track facilities on piers and quays and also near-by, properly laid out, supporting yards for the storing and sorting of cars free from interferences with neighboring operations.

Several other features remain for brief comment. The unification of the management, under joint control, of rail and water terminals in the waterfront zone of each port is essential to a full solution of the port problem. With this, should go a co-ordinating belt line under the jurisdiction of the port management; an "open door" for the vast fleet of independent craft unpossessed of exclusive facilities—leased or owned; and working arrangements with the trunk lines, whereby freight consigned to the port shall be suitably grouped at distant clearing yards for movement direct to the particular location for which it is destined, without the need for intermediate reclassification or breaking bulk. There is much room for improvement through a further restriction of free storage which is now a wasteful burden on the rail carrier. Ample warehouses and open storage areas adjoining the piers and quays, with moderate charges, would go far to decrease this evil.

The argument so far has been from the standpoint of international trade. It should not be forgotten that whatever is done to increase the terminal efficiency of American ports may perhaps some day save the National existence when the stress of war may again tax the ports to their utmost. The lessons of the World War should prompt Americans to cure their ills in time of peace. Not to do so, at a modicum of the cost of a few $40 000 000 battleships, seems most culpable to one who witnessed the saving of Paris and, therefore, the cause of the Allies, through the presence of the outer belt line at that city, and who likewise saw how close the port of New York came to a breakdown, due to congestion and inefficiency, during the darkest days of the war. No American would like to see repeated the predicament of Washington in 1776, when the loss of the Battle of Long Island found him with nothing but rowboats with which hurriedly to rescue his army.

Relief in all these particulars would appear to be hopeless unless some competent central governmental agency, clothed with power, shall vigorously take up the problem in all its commercial and military phases, with the determination promptly to find and enforce the application of remedies. It is idle to expect that the ship interests and the many railroads will make any substantial united progress in that direction. Under Section 500 of the Transportation Act of 1920, the duty is placed on the Secretary of War to investigate this

problem in harmony with the declared policy of Congress "to promote, encourage, and develop water transportation service and facilities in connection with the commerce of the United States, and to foster and preserve in full vigor both rail and water transportation." Along with this, it is the duty, in the interest of the country at large, of the Interstate Commerce Commission now called on to underwrite the return on railroad investments, to look into and correct matters that affect the operating expenses of the railroads which are burdened with much of the wasteful practices at the ports, from which waste, the water carriers, often flying a foreign flag, are reaping the benefit. Then, it should not be forgotten that, Americans as a people, are vitally interested in the provision of efficient ocean port terminals for their gigantic merchant marine, as to which it is the duty of the Shipping Board to take action.

Can the Society perform a higher public service than to press on the Secretary of War the need for moving promptly and effectively in accordance with the Transportation Act of 1920, so that henceforth joint water and rail port terminals shall have planned and constructed in a manner that will best serve the country from both the military and commercial standpoints, (1) wide piers for the closest possible liaison between land and water carriers; (2) the best of mechanical devices for the economical and speedy transshipment of freight; (3) unified management of all transportation facilities within the water-front and contiguous zones at each port, including a co-ordinating belt-line railway; and (4) co-operation between the trunk lines, the ship interests, and the port managements for cheapening and expediting the interchange of freight and passengers at the ports? By seizing this oppor.unity for public service now, engineers may be instrumental in forestalling the future creation of port facilities on plans adverse to the true interest of the Nation.

In order that such a movement may be started, the speaker respectfully suggests to the Board of Direction of the Society that the question of the relationship of water and rail carriers at the ports be taken up with other National engineering societies, with a view to its forceful presentation, primarily to the Secretary of War, on whom the responsibility of investigating this matter is placed by law, and, secondarily, to the Interstate Commerce Commission and to the Shipping Board.

PORT ADMINISTRATION

By B. F. Cresson, Jr.,* M. Am. Soc. C. E.

Great ports cannot reach the condition of maximum efficiency, without some central co-ordinating authority to direct them, any more than any great business enterprise. It is no more reasonable to expect that a port can develop and function properly by leaving its development and operation in the hands of transportation companies, than to expect that the Pennsylvania Railroad could be operated efficiently with a separate board of executives and a separate policy in Philadelphia, in New York, in Chicago, and in Pittsburgh; nor if that great system had separate executives to handle its line haul, yard operations, passenger business, freight business, freight-houses, and passenger stations.

A central directing body is as necessary to the proper development of ports as a body of directors to the development of a railroad system. Without either, the situation is entirely against the modern principles of business; it is uncertain in its results and not in accord wth modern civilization.

Time was in the history of practically every great port when the movement of commerce was simple; when there were comparatively few carriers, little competition, and plenty of room to expand; when commerce was attracted by natural advantages; where ships could dock and transfer their cargoes to land carriers with the greatest ease. It made little difference physically with the business of the port in those days whether docks were built, or quays or moles constructed; whether piers extended into the fairway, or whether canals were dug into the land. The important matter was to get contact between deep water and land in the easiest manner.

Before the creation of the great transportation systems, before the advent of the railroads with their powerful interests extending throughout the backlands, before the era of modern finance and business, the need of port administration as it is viewed to-day, did not exist.

In the days of ancient Tyre, land transportation was by wagon, or by skid drawn by man-power, horse, or oxen. The ships that sailed the seas were driven by the wind or by man-power at the oars. The cargo was handled into and out of the ship by men at windlasses; the situation was simple.

These conditions obtained, generally, at Venice, a great port in its prime, and at Amsterdam, which, prior to the supremacy of London, was an all-important port. These ports were laid out for sailing ships and for river and canal barges, and, even in London and in some of the older ports in Europe and in the United States, the early facilities as created were for sailing-ship service, without any thought or design for the service of railroads or of freight-handling machinery.

The situation is now far more complicated. With the advent of steam, it was possible to increase the size of ships, their speed, and their equipment. With the advent of the railroads, there arose the necessity of contact between their tracks and the ships. With the growth of machinery, a different design of port facilities became desirable. With the very many interests—financial,

* New York City.

transportation, operating, political—it has become a prime necessity to have some powerful authority to co-ordinate the facilities and the operations, and it is now an established fact that there must be public control which should extend not only over the harbor waters, but over the lands and the facilities adjacent thereto.

A modern port is made up of a multitude of different operating and administrative units. There are the transportation lines, the railroads, the steamships, the local marine operators, the river boatmen, the dockmen, the truckmen, the warehousemen, and the great financial interests directing private capital into profitable channels—all fighting for position—and not the least of these interests are those political. It seems impossible to divorce port control entirely from politics, and more than one port has suffered by political domination and by the ability abruptly and without redress to change the entire policy of its development and operation. The surest cure against political interference with the administration and development of the port is to appoint the commissioners in authority with overlapping terms of membership.

The public control by a central port authority does not mean the public ownership of port facilities. It does not mean that private investment must be converted into public holdings, nor does it mean that private initiative should not be encouraged and private money expended to produce modern terminal facilities. It does mean, however, that all these things should be harmonized by a degree of public control that will make effective a quicker and cheaper interchange between land and water carriers, will make the most advantageous use of the water-front and the waterways, and will weld the whole port into a terminal unit that will function with the highest degree of efficiency and economy.

The administration, ownership, and control of a port is not necessarily a part of the local government. There are many examples where the water-front of a city is entirely out of the control of the city itself and is under State or Government control, and, in many respects, this appears to be a right principle, as the port itself and the port facilities can be regarded in no case as strictly local facilities. They are in the service, not only of the city, but of the backlands, and, indeed, of a large part of the State and country tributary to the port.

The Port of Montreal is under the Dominion Government; the Port of Boston is under the Commonwealth of Massachusetts; and the Port of San Francisco is under the State of California.

In the Old World, London, is an interesting example of the trend of modernism in ports. For decades the greatest port in the world, handling the greatest tonnage, its supremacy was threatened by Liverpool.

The Port of London was under a number of private companies which owned the water-front and the terminal facilities and which levied from the commerce passing over the docks tolls sufficient to carry the investment and to yield a profit to the investors.

Liverpool was under a public trust made up largely of representatives of those companies using the port, and the returns from the use of the port were

needed only to meet the current expenses of running it, the interest on the public bonds issued create it, and the credit necessary to enlarge it.

After an examination by a commission and a recommendation, Parliament, realizing the necessity for maintaining the pre-eminence of London, not only created the Port of London Authority with jurisdiction over the port facilities of London, but also authorized the funds necessary to buy out the private interests and place the ownership of the Port of London under a public trust. That this has been successful need not be discussed.

In spite of the World War, London has been proceeding with its programme of creating modern terminal facilities, and under its central control of ownership and operation will develop and prosper increasingly as time goes on.

In the United States, as a rule, private interests have initiated the development of the ports. They have sought to attract and control commerce by creating good facilities. They have acquired and held strategic water-fronts, but with the development of the railroads, and with the growth of business, there has come an appreciation of the necessity for a very marked degree of public control over port development.

The waterways are the public highways to the sea, and, as such, it is to the interest of the public that easy access to them be provided. This can best be done without the necessity of paying private interests for the right of so doing.

Private enterprise, however, need not be abandoned; there is ample room for private terminal works at all the ports, and they should be encouraged. Private domination of the water-front, however, should not be permitted, but all private interests should be co-ordinated into a general plan, all lined up in their proper position, with their proper functions to perform.

This discussion would be incomplete without referring to the situation at the Port of New York.

The first ship sailed into New York Harbor nearly 400 years ago, the explorer, Verrazano, being the first known European visitor. More than 300 years ago, Hendrik Hudson sailed up the river that bears his name, and, in 1624, Albany was settled and New York City two years later.

A century ago, there were no railroads. Eighty years ago there were a few railroad lines running short distances back from the shore. The Erie Canal, commencing 100 years ago, virtually marked New York to be the country's chief port.

In 1914, more than 8 500 ships entered and as many more left the port. More than 45 000 000 tons of commerce entered and left the port during that year. More than 200 companies operate ships into and out of the port. The 12 railroads which reach it carried into, out of, or through, the port in 1914 more than 76 000 000 tons of freight. The value of the foreign commerce of the District of New York for the fiscal year of 1917, including gold and silver and foreign exports, amounted to more than $4 600 000 000.

The Port of New York lies in two States, New York to the east, and New Jersey to the west, with the dividing line down the Hudson River, down New York Bay, through Staten Island Sound, and through Raritan Bay. On one side of the port is the great city of New York which, for 50 years, has had its Dock Board or Dock Commission; on the other side, there are

fifty or more separate municipalities, and until 1921, there has been no central co-ordinating authority to aid in the development of the port as a unit.

It may be argued that this great business which the Port of New York has handled indicates the absence of any need for central control, but New York has a harbor of unrivaled excellence, with 800 odd miles of good water-front, with an absence of any tidal difficulties, and a noteworthy freedom from fog and ice.

New York in its growth and its establishment as the financial center of the New World, however, is overtaxing many of its port facilities, with the result that costs are excessive, that lack of any joint rail facilities has caused the use of a large amount of valuable water-front for rail purposes, and delays and congestion which have existed here, in addition to costs, have tended to divert commerce to other ports. New York is not selfish; it does not desire business which should go through other ports, but it aims to maintain its own position and to accommodate the commerce which would naturally flow through it by reason of its geographical and financial position.

Realizing the necessity for some form of administration that would encourage the better use of existing port facilities and the development of new ones along lines of greater co-operative efficiency, and after a careful and painstaking examination and study extending over several years, the States of New York and New Jersey enacted laws, and, on April 30th, 1921, representatives of the two States met and signed a compact or agreement, which compact or agreement created the Port of New York District, embracing all of New York City on the east, Yonkers, New Rochelle, and other communities; and on the west extending beyond Paterson, Passaic, and New Brunswick, N. J. This control extends about 25 miles to the north of the City Hall in Manhattan, about 16 miles to the east, 23 miles to the south, and 20 miles to the west.

By the same action which created the Port of New York District, the Port of New York Authority was created with broad powers, which powers can be exercised on the acceptance of a comprehensive plan by the Legislatures of the two States. This plan in conference with the representatives of municipalities, civic organizations, and transportation interests, is now being prepared for submission to the next Legislatures of two States.

This treaty or compact admirably expresses its purpose in the following words:

"Now, therefore, the said States of New Jersey and New York do supplement and amend the existing agreement of 1834 in the following respects: Article I. They agree to and pledge each with the other, faithful co-operation in the future planning and development of the Port of New York, holding in high trust for the benefit of the nation the special blessings and natural advantages thereof."

This compact or agreement between the two States has received Federal sanction by a joint resolution of the Senate and the House of Representatives in Washington, and by the signature of the President of the United States, on August 23d, 1921.

The Port of New York District and the Port of New York Authority are assured facts, sanctioned by the States and by the Federal Government; and thus, New York, the great port of the world in the volume of its tonnage and its business, has come to a realization of the necessity for port administration extending over its port district, and, in addition, will become able to handle more commerce, more rapidly, more cheaply, and more efficiently.

PIER DESIGNS AS DEVELOPED FROM QUAY DESIGNS

By H. McL. Harding,* Esq.

The purpose of this discussion is to determine the minimum width of piers whereby a constant uniform flow of miscellaneous cargoes through the pier may be accomplished without congestion.

It is generally accepted by terminal engineers that for the discharging and loading of freighters there is greater speed with economy at quays than at piers. Where there is ample room, and the physical conditions are favorable, it is desirable first to construct quays and, when more berths are required, to project piers. For an equivalent time-transferring capacity, a quay will require an investment of much less than half that of a pier.

There is more or less uniformity in the plan design of quays for ocean and Great Lake freighters. Each quay unit of a quay terminal is a little greater in water-frontage length than the longest vessel that may berth there. For ocean freighters this may be taken as 700 ft.

For each 700 ft. of quay length, the superstructure facilities consist of railway tracks, dray ways, paved, open dray areas, two sheds, overhead cranes with traveling conveyor hoists, and various surface carriers.

On this quay unit of 700-ft. length, the two sheds, each 200 ft. long, are placed, leaving 100 ft. of space at each end of the unit and 100 ft. of open space between the sheds. These sheds have a temporary holding capacity of 600 000 cu. ft. gross and 400 000 cu. ft. net, and are covered working areas, only for assortment and distribution, freight not removed within 48 hours being transferred to the warehouse.

It is necessary to have a supporting warehouse into which the goods may be placed at the expiration of 48 hours, the expense of moving to be charged against the goods.

Quay units may be taken as 150 ft. in width which is divided as follows: From the edge of the quay wall to the shed is 50 ft., with two railway tracks; then the shed, 50 ft. wide; and 50 ft. to the rear of the shed, with three railway tracks, making in all five railway tracks per unit. The outer 25 ft. in front of the shed is for dray ways and ship-side approaches and the remaining 25 ft. for the two railway tracks. The width of the shed may be increased to 70 ft., but, with steam railway practice, the narrower width is preferable.

The roof surface which is used chiefly for transfer and not for storage is of sufficient strength to sustain a load of 250 or 300 lb. per sq. ft. and is of sufficient height to admit of 20 ft. of tiering. Racks are often used in high tiering. At Manchester, England, freight is tiered as high as 40 ft. On account of the tiering which is done by overhead cranes and traveling electric hoists and tiering machines, it is necessary to reserve less floor space for longitudinal and transverse movements, than where there are chiefly floor movements. The expense and time for long horizontal floor movements are greater than for short mechanical vertical movements. There are a number of fixed overhead crane tracks perpendicular to the length of the shed and

* New York City.

on these and on the movable cross-tracks travel overhead hoists. These traveling hoists may be operated either from cabs or from the floor.

To the rear of the sheds the three railway tracks are spanned by gantry cranes similar to those in the front of the shed, unless warehouses are constructed, in which case standard bridge cranes supporting revolving gantry gib cranes would be used.

In the 300 ft. of open space on this 700-ft. length of quay is placed bulk freight, lumber, steel girders, and freight which is not affected by the weather. It is not desirable that such freight should pass through the transit shed, or that the expense and delay of lightering or of moving the ship to another berth should be incurred.

Similarly, in other mixed cargoes of which the greater part may be out-of-door freight, an open quay has been deemed necessary, even though there may be quite a volume of general merchandise for which there must be a rehandling. By the use of the open spaces described, the necessity will be obviated at a public terminal of open quays especially where the cargoes are generally composed of different commodities. Greater speed and economy can be obtained by utilizing these open spaces instead of sheds. There are also fire-risk advantages in not having the sheds continuous over the whole area.

The warehouses of which there may be a large number are not included in the quay unit width, although they are indispensable to any successful terminal.

The aforesaid plan description and the enumeration of the mechanical and other facilities of the quay unit are given in order that it may be understood how the pier is derived from the quay.

The capacity of the quay unit should be measured not by the storage capacity of the unit shed and open spaces, but by the rate of flow of the freight through the unit. The reservoirs for the storage of goods are the warehouses, not the transit sheds. The rate of flow is preferably figured by cubic feet per unit of time, hence the rate of flow is controlled by the volume, not by the weight totals.

A 7 000-ton ship has a net registered capacity of 700 000 cu. ft., or 100 cu. ft. per ton. This 100 cu. ft. per ton is taken as the unit of flow measure instead of the 40 cu. ft. per marine ton, which is much below the average of general merchandise. The 100 cu. ft. per ton is chosen to cover average maximum conditions.

To determine what maximum rate of flow may be required, this 7 000-ton ocean or Great Lakes freighter may have five hatchways or continuous openings. Each hatch will have two winches and in some cases four winches and two cranes. There will be, therefore, ten winches and ten cranes for each ship operation.

The function of the ship's winches is to draw the freight from between decks above the upper deck, where the draft is burtoned to the hook of the fall rope of the gantry crane and swung by the crane to the shore. In loading the ship, winches are used in stowing.

For example, assume a low average of only 60 cu. ft. per draft (1 200 lb.) in discharging and 60 drafts per hour per crane, which would be 36 000 cu. ft. per hour, making 360 000 cu. ft. per day of 10 hours, or 720 000 cu. ft. with two shifts. Although this 60 cu. ft. per draft may seem to be a low average, it is desired to make the estimates conservative.

There is a net holding capacity in both sheds of 400 000 cu. ft., or about 1 day's flow from the ship, and, in addition, about 250 000 cu. ft. in open spaces for coarse freight. The 650 000 cu. ft. makes a wide and deep channel for the flow. To prevent congestion it is necessary, however, that a portion of the day's flow from the ship be kept moving as much as possible continuously through the shed.

A certain quantity of the inbound cargo will be discharged over the offside of the ship on harbor craft, another portion may be placed directly on cars or other land carriers, and some also will go directly into the warehouses.

The volume of each draft might be taken as 80 cu. ft. instead of 60 cu. ft. There will be variations due to the character of the cargo, but from 60 to 80 cu. ft. is a fair average.

It is evident, however, that provision must be made to care for a flow of from 36 000 to 48 000 cu. ft. per hour, or 360 000 to 480 000 cu. ft. per day of 10 hours.

This, however, represents the flow capacity for only one ship at this berth; hence, the quay dimensions of 700 ft. by 150 ft. should take care of the flow of from 360 000 to 480 000 cu. ft. in 10 hours, but with little surplus capacity.

If sheds have less height and a greater floor area, there should not be a gross capacity of less than 600 000 cu. ft. In order that the freight movements of lifting, tiering, and depositing within the covered areas may be done mechanically, one story is preferable to two, since there will be required only one overhead installation of machinery, while there would be two for a two-story shed, besides twice the superintending force. Where there are two or more stories, much of the transferring and handling is performed by manual labor.

At Manchester, England, the sheds have a clear height of 50 ft., which with the same ground area would give a capacity of 1 000 000 cu. ft. Freight can be taken down and transferred to any portion of the Trafford Estates (Manchester) for 12 cents per ton, which denotes great speed of movement.

Different figures may be deduced, but those mentioned will form a fairly close average for high speeds of transference and handling by machinery. Sufficient cranes and traveling hoists can be provided to keep up the flow from the gantry jib cranes. If, however, back of this quay there should be dredged a waterway and a berth provided for another ship of similar capacity, there is no doubt but that so much cargo flowing on this 150 ft. of width from the ships would result in delay, congestion, and the too often unnecessary detention of the ship.

Also, it is evident that on a pier only 150 ft. wide and 700 ft. long, in the accommodation of two ships, one would be far too narrow for rapid and economical conditions. The 150 ft. therefore, may be regarded as the minimum width of the quay for one full-length ship.

If two sections of a quay, with all facilities and equipment, are placed back to back giving a width of 300 ft., results could be obtained from this pier which would be nearly equivalent to those obtained from two quay sections each 700 ft. To express the plan in another way, one quay section may be regarded as bent back of another quay section, each constituting one-half of the 300-ft. pier.

On a two-unit pier of 1 400 ft., it is evident that there should be four quay sections each 700 ft. long, two sections on each side of the pier, but that each working-pier-berth-section should still be 150 ft. wide.

It is evident that because the freight of the outer sections of the pier has to pass between the two inner sections, an additional strip in the center of the pier may be required. This strip may be of sufficient width for three additional railway tracks, based on the supposition that the six inner tracks of the 700-ft. pier are designed to be worked to capacity. Railway operators know the necessity and great advantage in having ample trackage, but 300 ft. is often of sufficient width for a two-unit pier.

It is to be feared that where in the past the piers have been too narrow, there will come an era when they will be too wide, regardless of the value of the water-frontage or of the investment required.

The following conclusions may be drawn from this discussion:

First.—Whatever the determined minimum working width of a one-unit quay section of one berth and with the capacity and facilities for one full-length ship may be, the pier for two ships should be double the width of the quay and have double the facilities.

Second.—It is desirable to have railway tracks and dray ways in the center of the pier to the rear of and between the sheds, as well as between the sheds and the water's edge.

Third.—Where a pier is several units in length, and provision must be made for through freight movements between the land ways and the outside units, such increase in width should be designed to be in the central space of the pier.

Fourth.—A pier may be regarded as two sections of a quay with all facilities, bent back to back.

DISCUSSION ON NATIONAL PORT PROBLEMS

By Messrs. William H. Adams, Arthur M. Shaw, T. F. Keller, Harwood Frost, L. F. Bellinger, John H. McCallum, Frank W. Hodgdon, M. G. Barnes, Nelson P. Lewis, and T. Howard Barnes.

William H. Adams,* M. Am. Soc. C. E. (by letter).†—The Great Lakes-St. Lawrence Tidewater Association is an organization of formal State commissions, created by Legislative enactment in the States for which the Great Lakes form the natural outlet. Its work is the organization of public sentiment and the creation of public demand that this improvement be made, by means of the assembly and dissemination of information as to the work involved and the advantages which will accrue to the nation and especially to the Great Lakes region. Its power is being derived from the rising pressure of public opinion which is being moulded, not by oratorical fireworks, but by a carefully planned campaign in which the ammunition consists of technical facts, presented in logical order to the world of business and trade. A large part of its work is being done through chambers of commerce which, to-day, are the most effective means by which business men can get action in public affairs.

In 1919 the President of the Detroit Board of Commerce was appointed on the Michigan State Commission and early the same year the Detroit Board constituted a new committee of the Board designated "The Inland Waterways Committee". The writer was asked to assume the chairmanship of this Committee and to form a working committee from the Board's large membership of more than 6 000 business and professional men. As formed, this Committee included engineers, transportation men, real estate men, and plain "business" men.

It was early seen that there would be an immediate connection between the National project for the opening of the St. Lawrence River to ocean navigation and the local problems involved in organizing the Port of Detroit to take care of world commerce. Accordingly, studies were begun and publicity planned to prepare the city for this phase of the matter. In the fall and winter of 1919-20 a series of articles was run in the *Detroiter,* the organ of the Board of Commerce, reaching its membership and also the business centers of other American cities. This series also appeared simultaneously in the *Michigan Manufacturer and Financial Record,* a financial paper with State-wide circulation. These articles described the development of port facilities in other harbor cities, including lake and world ports. They were carefully prepared and intended to be technically correct, but were written for the business man. They emphasized the business aspect of the work. They told of the relative importance of the "hinterland" as a possible market, and gave records of commerce running back several years. Where available, figures on the cost of port development were given, and comparisons were freely drawn with the facilities for commerce at Detroit. This series ran for about six months. The articles were illustrated, to some extent, and were widely copied in trade publications and

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* Detroit, Mich.

† Received by the Secretary, August 31st, 1921.

house organs. They inspired frequent editorial comment in Detroit and Michigan newspapers.

Sandwiched into this series was the National publicity for the St. Lawrence Project. All National meetings, conferences, and conventions were carefully reported. Each such event is in large measure a repetition of former meetings, that is, little that is new is brought out at any one conference, although the general mass of data may be constantly augmented. What is generally missed by the engineer is that, while another meeting may not bring out any new data, it has afforded a new opportunity for some one to express an opinion, or for a community or organization to take sides, consciously or otherwise. The data may not be, but the promulgation of it is, news, every time it is done.

Faithful following out of this principle results in much repetition of the same or similar information, but it is probable that each publication reaches some new readers. It is a well-known fact, moreover, that the average reader pays only partial attention to what he reads. If a certain address were to be given in ten consecutive weeks, a new press story could be written each time. The curious fact, to the engineer, is that the general public by no means realizes that it is the same address or even the same set of arguments. Only a small part of one's audience interests itself in "arguments" anyway, or could tell why it believes as it does. About all the average man on the street gets out of a press story is that "the well-known Mr. So-and-So addressed the Blank-Blank Society last night on 'Port Improvements at Detroit' ", and he says to himself, "What a wide interest there is being created in this matter. There must be something in it." Then, he begins to talk about it, and what is more important, he begins to study how it will affect his own future and, in Detroit, he lays plans to acquire some of this water-front while it is cheap, "which will certainly be valuable", he says, "when these hopes are realized". He finds that some of his present interests will be improved by increased commerce and straightway becomes a booster for the proposed enterprise, although he actually knows little about it that would interest an engineer.

In the spring of 1920 the Board of Commerce began an investigation covering in exhaustive detail the actual interest Detroit and Lower Michigan would have in the proposed ocean route. This involved a considerable personnel of field and office workers under a paid secretary and its results were embodied in a book "Detroit and World Trade".* This volume is actually Detroit's brief for the opening of the St. Lawrence River, and was presented at the hearing before the International Joint Commission in October, 1920.

During the summer of 1921 an important Conference on the St. Lawrence project was held under the auspices of the Detroit Board of Commerce. It was attended by representatives of cities and States from all over the country. A three-day programme was presented, almost entirely technical, covering all phases of the subject. The work of this Conference received wide comment, nearly ten thousand columns of press accounts being noted within the ensuing months. Favorable public sentiment was aroused to a marked degree. This Conference was followed by a storm of requests for speakers at public and semi-public functions throughout the State. Boards of trade in other cities

* Copy on file at the Headquarters of the Society.

called for addresses; the University of Michigan put on a lecture in its summer school; civic clubs, Rotary Clubs, Stationary clubs, women's clubs, engineering societies, labor unions, lodges, and churches—every organization which so multiplies—and divides—the activities of a large city, clamored for speakers.

Each such event was heralded in the press; each one furnished "copy" for a press story afterward. Of course, an effort was made to fit the presentation to the audience. The appeal to real estate men was not the same as the lecture to public school audiences. Such a subject has so many phases, however, that little difficulty is had in adapting the subject-matter to the time and the place.

Following this July conference, it seemed evident that public sentiment was sufficiently aroused so that the main project could not be seriously delayed or blocked. It was decided, therefore, to form a new group for more intensive study of local conditions. In preparation for this phase, the writer attended the Chicago Convention of the American Association of Port Authorities, contributing an address on "Detroit's Port Problems", which was widely printed in technical and business journals and, including illustrations, was generally published in the Detroit press. The new Port Development Committee of the Board of Commerce was formally constituted in January, 1921, with the writer as Chairman, and a small working committee. Engineering societies and the many business clubs of the city were invited to form their own port development committees, the chairmen of which would be invited to serve on the Central Committee of the Board of Commerce. In this way, the interesting discussions of the Committee were carried back to the other groups of business men.

The work of the Port Development Committee during 1921 has been largely clearing the ground for action. An "Act" to create a Joint Port District, including Detroit and a number of down-river municipalities, was drafted and made the subject of several conferences attended by representatives of all the districts involved. This was accompanied by much press comment, the "Act" being printed in full in most cases.

It was early found that constitutional provisions would prevent the organization of such an inclusive port district, and work was begun on a constitutional amendment to remove the ban. With much attendant press comment, this was pushed through the Legislature, at Lansing, and the campaign for ratification is now on. Work is also in progress on a comprehensive "Act" to incorporate the Port District of Detroit, looking ahead to the next session of the Legislature in 1922.

The interest which has been created is so great that there is constant pressure for more news stories and interviews from the press. The Port Development Committees of Detroit are probably averaging at least a column a week in the principal dailies of the city and State in support of port improvement and the Lakes-to-the-Sea Project.

At the request of the Detroit Board of Education, and following a lecture before the assembled principals and assistants of the public schools, a small textbook is in preparation for use in the Seventh and Eighth Grades and in

the High Schools. Debates are encouraged in High School circles, and data on both sides are supplied to contestants.

In conclusion, it should be mentioned that both Committees have had the aid of Mr. Tom L. Munger, a veteran newspaper man and writer, as Secretary. He can find "news" in what is to an engineer the baldest statement of well-known facts. Many of the simplest facts of engineering are absolutely unknown to the public. They only need to be related to some subject which has for the moment the public interest, and to be discussed in an interesting way to become "news", not once simply, but over and over again, as long as the public is still interested in the subject to which they may be, temporarily, related. A technical article, written for the engineer and published in an established technical magazine, will appear once only, be read and filed by a few engineers and forgotten. If written for the general reader and published in the daily press, or in business publications, it will be read by thousands and reprinted for weeks and months. It may become the source for numberless stories by "hack" writers to appear in quasi-technical publications for years. An address by the writer, delivered nearly a year ago, is this month (August, 1921), being republished in a British magazine on the other side of the world.

Finally, newspaper space should not be undervalued. One is tempted to value only the front page as worth while, but, from long experience, it is evident that even the smallest inch of space on the least important page of the daily paper, is read by thousands. There has never been mention of the Port Development work in the daily press, even in the most casual manner, which has not been brought to the writer's attention by readers, many times.

Even the foregoing discussion, void as it may seem of interest to the general reader, will be the subject of paragraphs in the press calling attention to the need of Port Development in Detroit, the fourth city in the United States.

ARTHUR M. SHAW,* M. AM. SOC. C. E. (by letter).†—In this discussion a far-sighted policy is advocated, of including in port plans, provision for "secondary" works such as railway terminal and classification yards, belt-line connections, warehouses, etc., and an attempt is made to show that such facilities, operated as a part of the port works, will assist in avoiding congestion and reduce the delay of railway equipment and of ships.

Three cases are cited by the writer, which show lack of space for such facilities as are suggested, one representing the maximum in present American port development, one presenting more nearly average conditions, and a Central American port which is just beginning to be a factor in world commerce. These three specific cases are chosen as representing conditions which may be found in most American ports.

For commerce to achieve the highest possible degree of efficiency and the greatest growth, facilities must be provided for the prompt, economical, and orderly transfer of commodities from the point of origin to the point of destination. In an effort to improve conditions, a study of the causes of delay, excessive cost, and confusion is most illuminating. Such conditions

* New Orleans, La.

† Received by the Secretary, September 8th, 1921.

are not especially apparent during periods of normal or light business, but become painfully obvious when an unusual or abnormal demand is made on transportation systems and port works. The war period was, perhaps, too abnormal to serve as a safe index, but the failure to achieve either speed, economy, or orderliness in the handling of the domestic and foreign commerce of the United States during the year that followed the signing of the armistice, would indicate that a study of the underlying causes of the most glaring defects might now be profitable.

The most obvious causes of delays to American commerce during past periods of unusually heavy business may be given as follows:

1.—Limitations of inland transportation systems, due to shortage of rolling stock and other causes.

2.—Congestion at terminals.

3.—Lack of wharfage and lighterage facilities.

4.—Shortage in the supply of ships.

As it is understood that this is to be one of a number of discussions, each of which will deal with some phase of port development or ocean commerce, the writer will confine himself to a discussion of the second point mentioned, although it is apparent that this has a direct bearing on each of the other three points. For the purpose of illustration, as already stated, three ports have been selected, representing what may be considered as the extremes and the mean of American port development, namely, the Ports of New York, New Orleans, La., and Puerto Cortez (Spanish Honduras). New York is far in the lead of all other ports of the New World in volume of business handled, and the list of commodities passing through this port includes practically every article used by civilized man. New Orleans is the second port of the United States in volume of business, but more nearly represents conditions as they exist in a dozen other ports of approximately the same rank. Its business is largely in agricultural products and in oil in bulk. Puerto Cortez has served as a port ever since the landing of the Spanish conquistador from whom it derived its name, but, until the last few years, it has not been prominent commercially.

At each of the ports mentioned, there has been serious congestion at times of heavy demands or there is evidence that such a condition will arise before any great increase in business handled may occur. Most great ports are the product of years of evolution, and any re-designing to meet new conditions must be made subject to the limitations imposed by existing improvements, many of which are not directly related to the functions of a port. On this account, especial (and usually expensive) methods are necessary to secure room for expansion and for the construction of auxiliary port works.

The following is a description of conditions which particularly affect expansion at each of the ports mentioned, with suggested means for overcoming some of the existing or prospective causes of congestion.

Port of New York.—This port has a great water-front, a spacious and well-protected harbor, and some of the most improved facilities for the storage and handling of cargoes. Tributary to the port are some of the most highly

developed transportation systems of the world and, still, extreme congestion at the terminal yards, warehouses, and wharves occurs at times of heavy movement of freight. If additional facilities could be provided for handling rail shipments and for the temporary storage of goods awaiting transportation, a great saving in handling would be effected and the lay-over time of railroad equipment and of ships would be reduced. This increase in facilities would include a belt-line railway connecting with as many trunk lines as practicable, railway classification yards, warehouses with both rail and water connections, and possibly sites for certain classes of industries which can be operated to the best advantage in the vicinity of large centers of population and convenient to facilities for inland and ocean transportation.

Along and near the water-front of New York City, there is no tract of land now available for such development as proposed herein, but, disregarding arbitrary State lines and considering as a logical part of the Port of New York, all that territory which might be used to advantage in the development of the port, there is still available (as far as any prior use is concerned) great areas of land almost ideally situated for the improvements suggested. These lands are known locally as the Jersey Meadows. The writer is not posted as to their present ownership, although it is assumed that they are controlled by individuals or private corporations and that, under proper procedure, they could be acquired for the purpose of public improvements. It is true that the utilization of these areas for the purposes mentioned would require certain improvements such as drainage and protection works, but no engineering problems would be involved, which have not been solved in a number of similar instances. These improvements will doubtless include the erection of levees to prevent tidal overflow, the cutting of diversion ditches to care for the drainage of adjacent high lands, the construction of a complete artificial drainage system (including sub-drains and pumps), and the filling, by hydraulic dredging, of certain limited areas. The drainage and protection of the City of New Orleans, and of various agricultural reclamation projects in the tidal swamps of Lower Louisiana, may be cited as successful examples of artificial drainage under similar conditions, although both the height of storm tides and the rainfall of the Gulf Coast section greatly exceed those of the New York district.

It may be of interest to note that, during certain years, more has been spent per acre in some counties of New Jersey in an effort to control the mosquitoes in the swamp lands than is required for the maintenance of drainage and protection works of a well designed Louisiana reclamation project. As an incident to the utilization of the Jersey Meadows for the purposes mentioned, a permanent solution of the mosquito problem would be reached.

Port of New Orleans.—The congestion at this port has not become as acute as at New York but, in many ways, the problems are similar. The Mississippi River serves as a natural harbor and affords deep-water wharfage for many miles along the city's front, but the demand for additional space is already insistent. The growth of the city was first along the river banks, and existing improvements make the cost of extensive port developments to the rear almost prohibitive. Partial relief will be afforded on the completion of the new

"Inner Harbor Canal" which connects, by locks, with the river and which will add about 11 miles of water-frontage to the harbor. A strip 1 000 ft. wide, along each side of this canal, and extending for the greater part of its length, has been acquired for port purposes, but no provision has been made for securing additional lands which will be required in the efficient operation of this element of the port.

The 1 000-ft. strips referred to should be used for wharves, wharf sheds and warehouses, and for delivery tracks, but, supporting these facilities, should be distributing yards and other improvements of the nature of those suggested for the Port of New York. Adjacent to the canal are many acres of unimproved or slightly improved lands which could be acquired at this time at a cost that may be considered negligible in comparison with their ultimate value to the port. The city is now served by a publicly owned belt railway which could be extended to afford all the service required for this new development.

Unless a far-sighted policy is adopted, which will provide for the early acquisition of vacant lands in the vicinity of such cities as New Orleans and New York, for the purpose of port development, it will be only a few years when other interests, not directly concerned in commerce, will secure control of the most favorably situated tracts and make more difficult the improvements which are so obviously required.

Port of Puerto Cortez.—This is a port now "in the making", although local tradition maintains that it was the first continental port of call of the Spanish conquistador, Hernando Cortez. Until very recently, the port facilities consisted of a commodious and safe harbor, a decrepit timber wharf, and a single-track railway leading to the interior of the country. Recent improvements include a reinforced concrete wharf, steel wharf sheds, mechanical loading equipment, and a convenient system of tracks for making rail delivery to ship side.

The Cuyamel Fruit Company operates from this port and has been instrumental in securing the improvements mentioned and also, with the co-operation with the Government of Honduras, is fostering a varied development of the interior. At present, it is probable that bananas form at least 90% of the exports from this section of the country, but it is realized by those who are most interested, that a business built up on this one commodity is standing on a slippery foundation, and a substantial increase is now under way in cattle raising, sugar plantations, and general farming. It is reasonable to expect that within the next few years, the present facilities of this port will be outgrown. Ample opportunity exists for the extension of the wharves along the harbor front, but of the "hinterland", there is none.

The Town of Puerto Cortez is built along a single street which occupies the crest of a ridge of beach formation. This ridge carries the track of the National Railway of Honduras, a footpath on each side of the track, and, in a few places, is wide enough for buildings fronting on this "street", although most of the buildings rest on made ground or on "stilts". Street traffic is confined to pedestrians, wheelbarrows, and such vehicles (railway velocipedes, push-cars, etc.) as can be run on the railway track. There are no saddle or draft animals in the town.

Back of this strip of high ground is an "impenetrable" swamp, the word "impenetrable" being used in its ordinary sense which is generally understood as not applying to exceptionally hardy explorers or to almost any engineering party. The problem of expansion of the secondary port works and of the town itself can be solved only by the reclamation of adjacent swamp lands, as has been suggested for the Port of New York.

In each of the ports described, the need is shown of additional space for handling present business and for future growth. Exact parallels will not be found in other localities but it is believed that, in general, the problems are similar and that their early solution will contribute to the upbuilding of American commerce.

Unless their transportation systems and ports can be relieved of unnecessary handicaps, Americans cannot expect to compete successfully in the handling of the world's commerce. Obstructions not only affect adversely the American merchant marine, but also hinder the development of American industries and will constitute a real menace to American armies in time of war.

T. F. KELLER,* ESQ. (by letter).†—The question under discussion, "National Port Problems", is one, which when studied from the standpoint of one interested in the improvement of the water-front of the City of New York, brings forward the thought that that water-front is a National port problem. The volume of its business and its close relationship with the remainder of the United States makes the problem a National one.

The City of New York has laid down a comprehensive plan for the development of the Port of the City of New York. There has now been advanced a plan, and the initial steps are being taken, to construct a tunnel under the Narrows, between the Boroughs of Richmond and Brooklyn. When completed, this tunnel with its rail connections, will effect the entry of all the transcontinental lines into the Stapleton Development with its 2 000 000 sq. ft. of pier space and 26 000 lin. ft. of berthing space for overseas steamships, will also open up for intensive development the magnificent Jamaica Bay, and will actually bring the railroads to the ocean.

On the northern side of the city, Flushing Bay will be thus made ready for development, and the wisdom and necessity of deepening Hell Gate to 40 ft. at mean low water will have been proven. Incidentally, connection will be made so that the vast undeveloped shores of The Bronx, along the East River, will attract attention for steamship and industrial purposes.

This tunnel and its necessary connections will bring the railroads through the heart of the Greater City, will open for development the outlying sections, all on deep water, and will serve for all time to show that the City of New York with its responsibility to the Nation has not been remiss in developing its resources.

At present, the city has about 85 miles of berthing on the sides of piers. It can be said, without fear of successful contradiction, that the proposed tunnel, with its necessary rail connections, will add a similar amount of

* Chf. Engr., Dept. of Docks, New York City.

† Received by the Secretary, September 6th, 1921.

berthing to its facilities, bringing sea and land together, and when served with all the cargo-handling equipment demonstrated to be necessary and efficient, the City of New York will have played its part in solving the "National Port Problem."

The stimulus thus given by the premier port of the world cannot but help in urging water-front development of other American ports to the end that the rivalry created will place all these ports on a high plane of efficiency and will solve the National port problem.

HARWOOD FROST,* ESQ. (by letter).†—The writer will discuss herein only one of the many topics that come under the subject of "National Port Problems", namely, the necessity of the conservation of labor at terminal ports.

The term "conservation of labor" rather than "saving of labor", is used, as the mention of labor-saving is too frequently and too generally associated with the dismissal of men, the reduction of the force; and, as a consequence, the introduction of so-called labor-saving devices in any kind of a plant has been bitterly opposed by "Labor" under the mistaken idea that their use would deprive many of the men of their jobs. Facts, however, have not borne out this idea, but rather the reverse, as has been fully demonstrated by the history of printing as affected by the development of the linotype machine, and of many other industries as affected by similar time and labor-conserving equipment.

In the past when labor was plentiful and cheap, it was bought and sold as a commodity, with little thought of its conservation in the manner of the conservation of National resources, such as water power, forests, fuel, etc., and as little consideration was given to labor by the nation generally as an important part of its resources. "Labor" looked on mechanical material-handling appliances as enemies, and their introduction was always accompanied by fears of strikes or other labor troubles. With an increasing scarcity and cost of common labor, however, a feeling is developing that such machinery may be looked on as an ally, a means of reducing the drudgery of the laborer's work, of conserving his strength, of increasing his efficiency and enabling him to earn a greater income, thus, in turn, improving his social position, giving him a greater buying power in the commercial world, and generally elevating him to a higher plane of citizenship.

The problem of labor has always been closely associated with the equally important problem of the economic handling of materials; the two problems are of most ancient origin and go hand in hand. There is no doubt that the transport, lifting, and placing of the huge blocks of stone used in the construction of the great Egyptian Pyramids presented serious problems to their builders, or that serious problems were also presented to the builders of the wonderful aqueducts, temples, roads, and other monumental works of Rome and Greece. Yet, with only a rudimentary knowledge of mechanics, and by the use of the simplest mechanical devices, such as the lever, wedge, pulley, etc., those problems were met and successfully overcome. Go as far back as one will into the historic and prehistoric past, and evidences of these associated

* Chicago, Ill.

† Received by the Secretary, September 7th, 1921.

problems of labor and material handling are found. These problems have ever existed; their solution has formed the basis of comparative civilization; they have had their effects on history, on society, on industry; they have been the determining factors in wars, and have made and unmade nations; and they have regulated commerce on sea and on land.

The same problems of labor and of methods of handling materials still exist. They seriously affect the daily lives of the people; they make for success or failure in industries; and they regulate the prices of commodities. These problems have always existed; their solution only is different. The ancients solved their problems by the use of vast numbers of slaves, and time was no object. There was no thought of the conservation of anything. So with Americans, when labor was plentiful and inexpensive, there was waste and little thought of it as a commodity of value, to be conserved. With the tremendous drafts on the manufacturing and labor resources of the United States caused by the World War, together with congested terminals and transportation difficulties, however, the people were brought to a conscious realization of the value of labor as a basic commodity and, also, that in the utilization of labor-conserving machinery for handling materials, Americans had not kept pace with other countries. In order to keep costs within limits that would leave a fair margin of profit, whether in the handling of goods through a marine terminal or through a manufacturing plant, it was found to be absolutely necessary to conserve the available labor supply and to raise the efficiency of the man-power by the most extensive possible use of mechanical appliances.

Through the necessities of their overseas war operations, Americans have suddenly developed into a more or less maritime people; they have greatly increased the number of their terminal ports; they aim to see the American flag flying on American ships in all parts of the world; in short, the war forced them into the business of ocean transportation and they now desire to compete for the world's trade against older and more experienced maritime nations. They, however, continue to ignore at their terminal ports the very methods and practices that brought about the efficient and profitable operation of foreign ports, and this in the face of the fact that these same foreign ports are gaining a certain amount of their maritime strength and advantage at the expense of Americans, that is, by the use of large quantities of time and labor-conserving equipment produced by American brains and industry. America has invented, and manufactures more modern freight-handling machinery than any other country in the world and uses comparatively less.

There are a number of reasons for this condition, which it is not the writer's purpose to discuss; he desires, however, to trace briefly one of the results of the condition. The waste in labor at American terminals in the handling of freight has been said to be more than $400 000 000 annually—it may be more or less, but, in any case, the figure is sufficiently high to warrant consideration and to impress one with the magnitude and importance of the problem of this wasted effort. This $400 000 000 of wasted effort may be looked on as representing at a fair rate of interest, a labor capital of $8 000 000 000 tied up by American terminal inefficiency; but, also, consider

that, in the passage of goods of any kind between producer and consumer, through the hands of jobbers, wholesalers, retailers, etc., any item of waste is multiplied by each middleman in just the proportion he increases his selling price above his cost, and that a waste at the source will cost the ultimate consumer about four and one-half times the original waste. On this basis, the $400 000 000 of unnecessary terminal waste would increase the cost of goods to the ultimate consumer by nearly $2 000 000 000. Further, since the finished product of one concern is very frequently another's raw material, the cost of production to the second concern is increased by the multiplication method with every handling, just as the price of commodities is to the consumer. This progressive increase in the cost of wasted effort most certainly has a serious effect on two of the most important problems of the day—the cost of living and the cost of production.

It is evident, therefore, that in the handling of the hundreds of millions of tons of miscellaneous freight which flow through the port terminals of the United States, wasteful methods add to the burden of the consumer, while quick and efficient methods reduce this burden.

The general public, however, does not seem to understand this. There are many men, business men, handlers of the essentials of life, as well as the consumers, who fail to comprehend why they should be interested in the terminal affairs of New York, New Orleans, or Seattle. They readily accept an increase in price said to be due to increased labor costs or increased freight rates, but they do not seem to realize that anything which affects handling costs in any one of the large terminals, or of goods anywhere in storage or transfer, becomes a matter of direct concern to each and every one of them, because they have not given consideration to the close and vital relations that exist between general business welfare and transportation service, that is, the various factors involved in the distribution of commodities. Nor do they realize the fact that whenever the cost of any single element of transportation, such as the unloading of boats, piling, or otherwise moving, of goods in storage, or the loading of trucks, rises above a fair normal level, it becomes a tax on business generally, and the ultimate consumer must pay for it.

The elimination of wasted effort in the handling of merchandise at terminal ports is a matter of public concern, it is a problem of the whole people; it is a National problem, affecting every one, in every locality, in every station of life. This burden of unnecessary taxation and waste cannot altogether be eliminated, but it can be materially reduced by the substitution, whenever possible, of machines for men, and thus by reducing this "cost of lifting", the "cost of living" also will be materially reduced. Any general reduction in the cost of moving goods through the terminals is certain to have its effect along the entire line, and the general public, like the ultimate consumer, is certain to share in the savings, as neither the middlemen, nor the transportation companies, nor any other single interest, can appropriate it all.

L. F. Bellinger,* M. Am. Soc. C. E. (by letter).†—There has been much in the newspapers recently concerning the old DeWitt Clinton train with its

* New Orleans, La.

† Received by the Secretary, September 9th, 1921.

diminutive locomotive and passenger cars of about one-twentieth the weight and capacity of those of the present. This train is really interesting to the present generation, because of the small size and small capacity of the passenger coaches and freight cars, namely, twelve passengers per passenger car inside, and only a few thousand pounds per freight car. At present, it would seem to be impossible to develop economical transportation with the numerous carriages of the DeWitt Clinton size required for a given volume of freight. There are many indications, however, that the container or the original package is to be reduced to approximately the size of the DeWitt Clinton freight car. Steel containers of approximately this size are used in the Warrior River Barge Transportation Service, and wooden containers are used to some extent on the railroads. These containers are taken from barges, lifted bodily by derricks, and placed on flat cars which make rather economical the interchange between barge and railroad train. Especially in less than carload lots, freight consigned to individuals is easily stored in these cubicles. If their total weight is limited to about 5 tons, the entire cubicle can be lifted from the flat cars and placed directly on the ordinary 5-ton automobile truck, delivered to the consignee, unloaded, and the container returned to the railroad.

The economical handling and storing of these cubicles is a matter of detail. At present, much of the package freight is unloaded by ordinary hand trucks, which will carry from 50 to 500 lb., from the freight car to its proper place in the freight-house, thence by hand trucks to automobile trucks, thence to the consignee's storehouse. As now used, the cubicles, with proper design, can be handled by motor-driven piling and stacking machines, carried from the freight car, and placed wherever desired in the freight-house if the cubicle is placed on skids in such a way that the stacking machine can take hold beneath. With the cubicle and the motor-driven stacking machine, one man can handle 5 tons on one trip as against the ordinary 500 lb. maximum with the hand truck.

The devices outlined make for cheaper transportation from point of origin to ultimate destination to the consumer. Heretofore, the effort has been made to reduce the cost of rail transportation per ton-mile until the ton-mile cost in the United States is reduced to its lowest limits, provided the large and expensive freight cars are fully loaded. By means of the cubicles a cheaper freight car is used, namely, the flat car, and the cost of transportation, especially through the freight-houses, is very much reduced.

The connection between the preceding discussion and the increase of port facilities in the United States is not at once apparent. Just as the intention in the past has been to reduce the cost of freight per ton-mile, so the intention of port engineers in the past has been to have a port which would be capable of easily receiving the largest deep-draft steamers that have been built. In order to accommodate these much advertised maximum sized ships, each small port has dreamed of increasing its facilities so that a ship of maximum size could lie at its wharves and, in the immediate future, rival New York in its commerce. It is believed that the time has come to take up the question of the use of the shallow ports of the globe, to utilize them for the benefit of the

remainder of the world to distribute the traffic which is now so concentrated in the deep-draft ports, and thus to reduce the cost of package handling which results in excessive terminal charges and adds so largely to the total cost of distribution paid by the ultimate consumer.

It is well known that the cost during the time a ship is in port is a very material fraction of its cost of operation. It is also well known that any reduction of time in port adds materially to the earning capacity of the ship. To a certain extent the smaller the ship the shorter is the time to unload and the less idle time is necessary in port. Reduction in the size of the ship can be carried so far that the demurrage is small, but the earning capacity of the ship while in transit reaches the vanishing point. On the other hand, the very large ship wastes so much time in port and, at present, requires so long a time to secure a full cargo that the ship of maximum size is now a losing proposition, which is also true of the very small ship.

There are numerous landlocked bays and harbors in the world, which will permit, with very little or no development, barges and ships of drafts of 10 ft. or less. These harbors were of value about the time of Columbus, or earlier. At the present time, they are unused because of the lighterage costs which occur in handling freight with hand trucks or on the shoulders of stevedores. The freight is handled from the freight warehouse ashore to the barges and again from the barges to the holds of ships lying off shore. Thus, the cost of transportation between point of origin and point of ultimate destination is made too great. There are a number of ports in the world which have spent millions of dollars to secure channels and slips alongside piers that will permit of ships of a draft of 30 to 35 ft. or more. The millions of dollars spent in securing facilities for ships of maximum draft come from the ultimate consumer indirectly in the form of taxes, but, nevertheless, it adds to the actual cost of transportation between the point of origin and the point of ultimate destination.

Instead of increasing the depths of channels, instead of increasing the depths at piers, and in spending additional millions for port facilities, it is believed the time has come again to consider the ports which will accommodate ships of medium draft, and to rehabilitate old grass-grown piers of ports which, although not dead, are at least in a comatose state. There should be a distribution of traffic from the present few large deep-water terminals to water terminals of medium depth, just as at present there is a change in the development of a city from the one large loop district, or the single business center, to several well distributed so-called "civic centers", each fairly complete in itself. When this distribution is effected the following advantages are found to exist: Low, fixed, capital charges; low terminal charges; low cost of land; low cost of building materials; low house rents; and low cost of distribution of food products to these small cities, all of which result in a low cost of living. Each advantage in itself bears directly on a lower cost of labor for those living at the "civic centers", or at the water terminals of medium depth.

Wherever there is a low cost of labor and a low cost of building materials there is a chance to utilize, by means of the cubicles mentioned, a low cost of handling exports from small ports through the small or medium sized ships

to the consumers in various parts of the world. The utilization of the medium sized ships through the medium sized ports will conduce to low freight costs from the point of origin to the point of ultimate destination, through the low costs of handling freight at such ports.

Any reduction in cost of production or of transportation will aid in stimulating trade between Europe and the United States. Any stimulus to trade accelerates the circulation of commercial products and gives debtors a chance to pay their debts and to increase the wealth of the world available for business extensions. Furthermore, it conduces to that international good feeling so necessary to restore business confidence and business health. Anything which will relieve Europe of its sufferings from business "gout", will loosen up the "rheumatic joints" of business in the United States.

It is believed, therefore, that attention should be given not so much to expensive port development involving millions, but to the increase in the use of the medium sized ports which will require comparatively little capital and yet will enliven many different sections of the body politic now in need of resuscitation rather than rehabilitation. In a country the inhabitants of which have a steady income, although small, the people are far more satisfied than those of a country or city where high priced wages obtain, but where work is intermittent. The idle time is used by Satan for the purpose of stirring up dissatisfaction, destructive criticism, strikes, and ultimate revolution.

John H. McCallum,* Esq. (by letter).†—The writer is deeply interested in all port development, especially as it affects all ports of the United States.

As the Port of San Francisco is the gateway to the Orient, the California State Harbor Commissioners are very desirous of furnishing it with the most up-to-date and economical equipment and devices for handling such commerce, and any information that can be gathered which would assist them in furnishing such facilities and equipment is eagerly sought. Those who are actively engaged in commercial pursuits and who use this port are of the opinion that the facilities are quite up to date.

The writer has been a member of the State Harbor Board for the past ten years and is quite free to admit that whatever success has been obtained for the Harbor of San Francisco is due largely to the spirit of co-operation between the members of the Board and all factors interested in the port.

During the period of reconstruction which began in 1911, the engineers of the Harbor Board were assisted very greatly by an Advisory Board of Engineers of large experience in water-front construction. In 1917, the rules and regulations, including the system of port revenue and penalties for the use of piers beyond the free period, were completely changed, this change being worked out with the co-operation of a committee appointed by the Board, consisting of those vitally interested in the commerce of the port. The new system has worked splendidly, and especially as it applied to penalties, for, as this was in a great measure the suggestion of the users of the port, they were bound thereto. In 1917, the Board also appointed an Advisory Committee of twenty

* President, Board of State Harbor Commrs., San Francisco, Cal.

† Received by the Secretary, September 6th, 1921.

men doing the greatest business through the Port of San Francisco to advise with it from time to time on vital problems of the port. This Committee is composed of importers and exporters, ship-owners and operators, representatives of the Chamber of Commerce, Warehousemen's and Draymen's Associations, the railroads, etc. The services of this Committee have been invaluable to the Harbor Board, and its unselfish devotion to the interests of the port has been wonderful. The writer is a firm believer in the policy of consulting the interests that are using the port facilities as to its development.

One of the great engineering problems confronting San Francisco and the Bay region is that of constructing a bridge from the San Francisco side to the Alameda County shore. Some study has been given thereto and a report has been made, but the problem seems to be still unsolved.

FRANK W. HODGDON,* M. AM. SOC. C. E. (by letter).†—Ports, as usually known, are points on the sea coast at which merchandise can be transhipped from land transportation to sea transportation, or *vice versa*.

In order to have a port, there must be merchandise to be transported and merchants to control it. If there is no merchandise, there can be no port, and if better facilities are furnished at another port, the merchandise will go there and the port with the lesser facilities will disappear as such, as has been the fate of Salem, Mass. In former times, vessel units and cargoes were small, and land transportation was adequate to convey any bulky merchandise only for short distances, so that many small ports existed which have now disappeared. As railroads were built and developed, they took up the distribution of bulky merchandise and delivered it far inland and brought to the sea the products of the interior. At the same time, the vessel units increased in size, with the result that the facilities furnished by the ports had to be radically changed and enlarged, and the assembling of cargo units could only be done economically by reducing the number of ports and concentrating the business at a few large ones at which the railroads could concentrate. This concentration will continue until the amount of business at any port exceeds its capacity, when it will become necessary to divert some of the business to other ports.

In general, merchandise will take the most direct route, other things being equal, but many things happen to change this, such as rates and facilities for handling the merchandise, congestion on the direct route, etc. The facilities furnished by a port are both fixed and variable. The fixed facilities are the channels, wharves, docks, warehouses, etc., the railroads and highways. The variable facilities are the vessels, railroad cars and equipment, and trucks. These are the facilities only, to handle the merchandise which is furnished by the merchants who are the real creators and life of the port and without which it could not exist. Usually, the port itself controls only the channels, wharves, docks, and, sometimes, some of the warehouses, freight-handling equipment, and railroad yard tracks on its own property. All the vessels, railroads, merchandise, and trucks, etc., are owned and controlled by others.

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* Boston, Mass.

† Received by the Secretary, August 31st, 1921.

From this it will be seen that the port merely offers an opportunity to carry on a business over which it practically has no control, except to offer the best practicable facilities to induce business to pass through it. The merchants control the business and can practically make or break any port.

Business like natural forces will follow the line of least resistance. A port naturally situated so as to offer the best conditions as to rates and facilities will secure the most business. New York is situated at the mouth of the Hudson River and, with this inland waterway supplemented by the Erie Canal, was especially advantageously located. It thus became the foremost port of America, attracting business from all over the country, even though handicapped by not having the opportunity to run its railroad trains on to the piers and thus be able to handle merchandise directly between cars and vessels.

The large volume of business and the low costs of land transportation induced by the low grades of the railroad and the competition of the inland waterways attracted more merchandise, so that vessels were attracted since they could usually find a cargo waiting, even if advance arrangements had not been made. At smaller ports, however, such spot cargoes cannot generally be found, and if a vessel brought a cargo to one of the ports without having engaged a return cargo in advance, it would have to sail in ballast and seek a cargo at some other port. This would increase the cost of operating the vessel, and this increased cost would have to be made good by increased freight rates when a cargo was secured. This would make the small port less desirable as a shipping point compared with one which could furnish spot cargoes at most times and, thus, again increase the desirability of doing business in the larger port. Large ports have more frequent sailings of regular lines and tramps than the smaller ports, and small shipments will find more frequent opportunities there and less likelihood of delays.

As already stated, in order to be successful, a port must be properly located, with efficient land transportation tributary to it and a back country capable of furnishing sufficient merchandise to load all the vessels frequenting the port and absorbing all the merchandise which is brought to it. The port must be well equipped with channels, wharves, docks, warehouses, etc., properly to handle the merchandise and must also have merchants ready to handle the business. A port, to be successful, must first have the business and the wharves, channels, and other equipment, and then the ships will come.

The various ports of the world are managed in a number of different ways: Private ownership including ownership by railroad and corporations; and public ownership such as State, city, or other political divisions. These different methods take no cognizance of the merchant who really controls the port by furnishing the business done through it, and who, if he had some interest in the management of the port, would have a great inducement to persuade other merchants to use it and, thereby increasing its business, tend to reduce the cost of sending his own business through it. Also, the merchants would be able to decide with first-hand information to just what extent it was advisable or profitable to install better or additional facilities.

The method adopted by the Port of Liverpool, England, one of the most successful ports of the world, is based on the idea that the merchant who

does business through the port is the person who should manage it. The Corporation of the Mersey Dock and Harbor Board is without capital or shares, and its credit is based on the prosperity of the port and the value of the property owned by the Corporation. The electors who annually elect the officers of the Corporation are the persons who, during the previous year, have paid port and dock charges of not less than £5. In order to be eligible to be a director, a person must have paid not less than £25. The Directors receive no compensation, but it is considered such an honor to be one that the office is always sought for.

By law, the port charges must be fixed to pay the interest, sinking fund, and maintenance. All improvements and extensions must be paid from the proceeds of loans authorized by the Government. The Secretary and Engineer of the Board are the paid executives. In this way, the merchants have control of the port, and it is to their interest to furnish it with as much business as they can, in order to keep the costs down, as the business is done at cost.

One difficulty at some of the Atlantic ports of the United States, is the inequality of rates when part of the wharves are owned by the railroads, and part by the city, State, or private owners. The railroad companies use their wharves as one of the facilities of the railroad, and make no charge either to the vessel bringing freight for the railroad, or to the goods which are sent over it, other than the regular freight rates. That is, there are no charges for wharfage or dockage, except on goods which are shipped to or from the wharf by other means than by the railroad which owns the wharf. Frequently, the charges for freight on goods brought to or from vessels at a railroad wharf, are less than on similar goods brought to the port for local consumption, although the terminal costs of the railroad are greater for the vessel, than for the local business.

Public or private wharves, of course, cannot compete with railroad wharves on their basis, and some means should be found, by which charges should be made, by which the actual cost of facilities at a port should be collected, on all goods passing through it, and, if possible, this should be made equal at all wharves. In order to do this, the whole port should be under one control and management, either public, or under public supervision. The exact form is not necessarily the same in all ports, but some form of operating the whole port, as a unit, seems to be essential.

M. G. Barnes,* M. Am. Soc. C. E. (by letter).†—Since 1914, measured by cost of rail transportation, interior points of the United States have been placed twice as far from the seaboard as formerly. At best, the wheat belt of the United States is handicapped in the world markets because of its great distance from the seaboard. Increased rail rates have aggravated this condition of affairs, lowered grain prices on the farm, and placed the American farmer at such a disadvantage as to limit his foreign market and, indeed, foreign grain may be delivered at the seaboard cities at a less cost of haul than from the grain belt of the United States. Since 1914, the cost of transporting wheat by rail to the seaboard has increased from 12 to 15 cents per

* Springfield, Ill.

† Received by the Secretary, September 6th, 1921.

bushel. The cost to domestic consumers in the Eastern States has increased to even a greater extent.

Based on the past ten-year average annual production of 794 000 000 bushels, these higher freight rates represent a tax on the producer and consumer of at least $100 000 000 annually. This would indicate that on a 10% basis, $1 000 000 000 might profitably be spent to reduce the cost of haul on wheat to the pre-war level; and, of course, if wheat could be hauled at pre-war prices, other commodities would profit in like manner.

Fig. 1 shows the location and quantity, or volume, of the surplus wheat of the United States, and its destination, the figures having been compiled from the records of the U. S. Department of Commerce for the past ten years. Although the destination of the export wheat is roughly accurate, the ports from which it is assumed to move are not the actual ones, but more nearly represent the economic lines of movement based on present rail-transportation facilities. During the open season of navigation, part of this wheat travels by rail and water to the Gulf, and a greater part by rail and water to the Eastern States and the Eastern seaboard, but probably not sufficient to change the routing suggested.

Various suggestions have been made for the improvement of transportation facilities and especially as it affects the farmer. The advocates of the St. Lawrence Ship Canal, permitting ocean liners to visit Chicago, Ill., Duluth, Minn., and other Lake ports, believe their waterway is the best solution of the transportation troubles and advocate its completion jointly by the American and Canadian Governments. Those who favor the improvement of the Mississippi River System of Waterways are equally sure that the export wheat should find exit *via* the Gulf ports. Others see relief only in the improvement of rail transportation.

A study of the wheat map of the United States (Fig. 1), may aid in solving this problem. The great surplus wheat States lie on the eastern slope of the Rocky Mountains, and the wheat finds outlet through four main depots—St. Paul, Minn., Omaha, Nebr., Kansas City and St. Louis, Mo. Under the present rail freight structure, the cheapest route to the seaboard is to the south through the Gulf ports. Shippers realize this, as shown by the wheat export figures for the fiscal years ending June 30th, 1920 and 1921. The shipments are:

	1920.	1921.
Through Gulf Ports......	97 000 000 bushels	160 000 000 bushels
Through Atlantic Ports..	79 000 000 "	92 000 000 "

This in the face of poorer facilities and shortage of boats at the Gulf ports: The Eastern ports also have the advantage of Lake transportation during part of the season. The increase in shipments through the Gulf ports in 1921 results from increased freight rates and the better supply of boats at these ports.

In any study of economic wheat transportation, one must not lose sight of the under-production that exists in the Eastern part of the United States. Turn, now, to the most economic method of supplying these needs. In export rates, St. Louis has the advantage over St. Paul of 10.2 cents per bushel

through Gulf ports, and Kansas City enjoys an advantage of 4.8 cents through the same ports. For domestic trade, St. Louis has an advantage over St. Paul in the New York markets of 12.3 cents per bushel (not shown on Fig. 1), but Kansas City and Omaha have an advantage of only 3 cents per bushel. St. Louis cannot supply the needs. Manifestly, the wheat should come from the St. Paul District, which it does very largely. Moreover, St. Paul is favored with a partial water route *via* the Great Lakes, and, in 1920, there was carried through the "Soo" Canal approximately the northern surplus shown in the Northern States of Montana, North and South Dakota, and Minnesota. Part of the "Soo" traffic, of course, was Canadian wheat. Flour shipped through the "Soo" Canals is annually about sufficient to supply the New York State shortage.

Next, in order, the shortage in the Southern States must be supplied. This can best be done by drawing on the surplus from Nebraska, Iowa, Illinois, Missouri, and Kansas, as shown. These States actually do supply much of this deficiency. This leaves, as exportable, surplus wheat grown in Wyoming, Colorado, Kansas, and Oklahoma, and the extreme Northwest, which properly finds exit as shown. So much for existing conditions. How shall we set out to improve them?

First, let us consider supplying the domestic trade. Reference to the map (Fig. 1), will show that the great deficiency exists in the Northeastern States. The domestic rail rate from St. Paul to New York is 36 cents per bushel, while the domestic rate from St. Louis to New York is 23.7 cents per bushel, giving St. Louis an advantage of 12.3 cents per bushel. This advantage can be more than offset by a greater use of the waterways now available. Large quantities of wheat and flour are transported annually over the Great Lakes from Duluth, Minn., to Buffalo, N. Y. The quantity of flour passing through the "Soo" Canal annually is sufficient to supply the deficiency in New York State. Now that the New York State Barge Canals are completed, shippers should avail themselves of this waterway to distribute wheat and flour from Buffalo eastward. The normal charge for transporting wheat from Duluth to Buffalo is 2 cents per bushel. At the present time, wheat is sent from Chicago to Buffalo at from $1\frac{3}{4}$ to 2 cents per bushel. In pre-war days, flour was shipped from Duluth to Buffalo for the equivalent of about 4 cents per bushel of wheat. In 1920, the rate was 6 cents per bushel. Wheat and flour from the North Central States would reach Duluth by rail at approximately 10 cents per bushel. This would permit shippers to deliver wheat and flour in Buffalo at rates of from 12 to possibly 15 cents per bushel. After it has reached Buffalo, it is in the territory of deficiency, and can be delivered by rail and water from that point. By an intelligent use of the existing waterways on the northern border of the United States, it is quite possible that a rate of 16 cents per bushel can be maintained from the St. Paul District to New York City, which would show an advantage over the St. Louis all-rail rate of approximately 8 cents per bushel. Navigation through the "Soo" Canal is carried on over a period of about 8 months per year. In order to secure the advantages of water transportation throughout the year, it would be necessary,

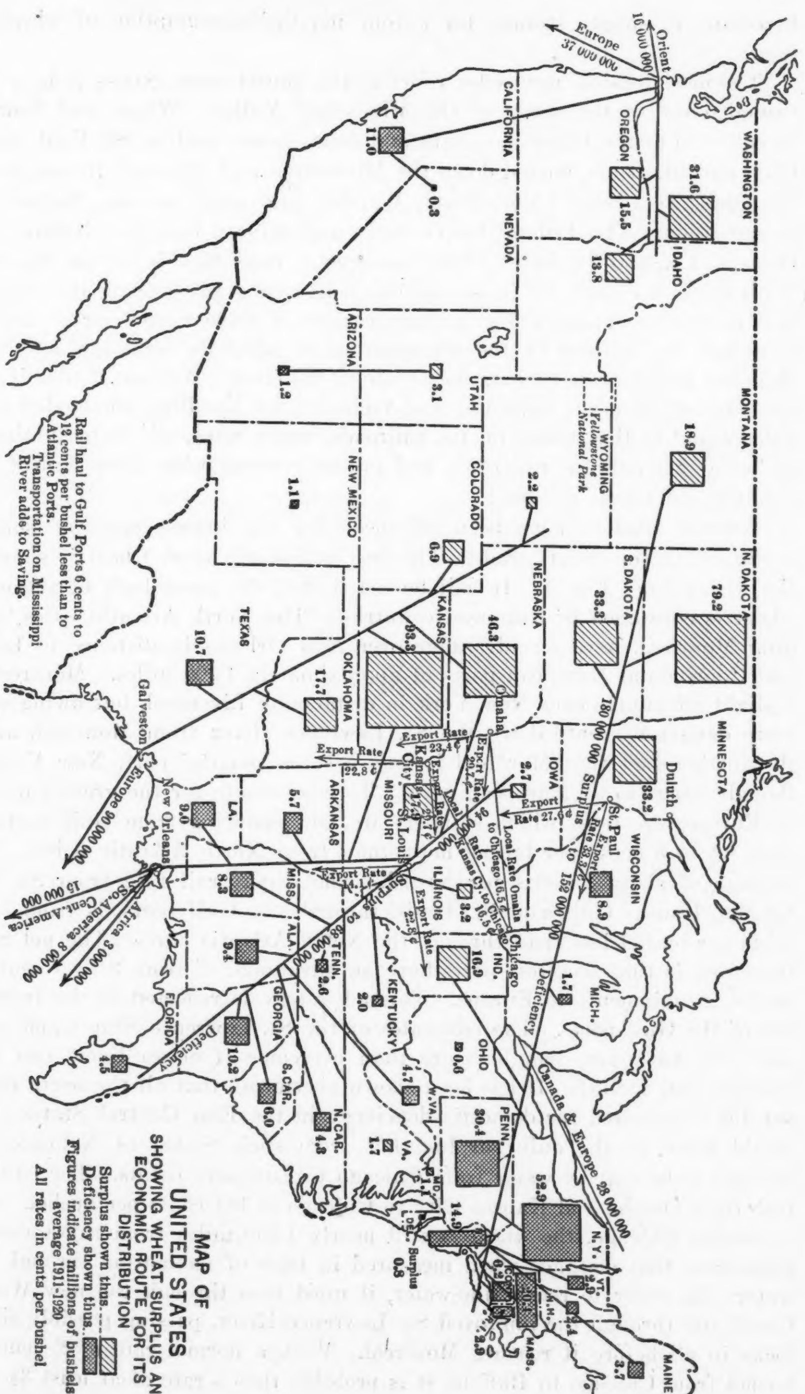
MAP OF
UNITED STATES
SHOWING WHEAT SURPLUS AND
ECONOMIC ROUTE FOR ITS
DISTRIBUTION
Surplus shown thus.
Deficiency shown thus.
Figures indicate millions of bushels
average 1911-1920
All rates in cents per bushel
Rail haul to Gulf Ports 6 cents to
12 cents per bushel less than to
Atlantic Ports.
Transportation on Mississippi
River adds to Saving.
Europe 37 000 000
Orient 16 000 000
180 000 000 Surplus to 152 000 000
94 000 000 Rate
Surplus to
Canada & Europe 28 000 000
Europe 90 000 000
15 000 000 Cent.America
So.America 8 000 000
Africa 3 000 000
Galveston
New Orleans
Omaha
St.Louis
Chicago
St.Paul
Duluth
Deficiency
WASHINGTON
OREGON
CALIFORNIA
IDAHO
NEVADA
MONTANA
WYOMING
UTAH
ARIZONA
COLORADO
NEW MEXICO
N. DAKOTA
S. DAKOTA
NEBRASKA
KANSAS
OKLAHOMA
TEXAS
MINNESOTA
IOWA
MISSOURI
ARKANSAS
LA.
WISCONSIN
ILLINOIS
MICH.
IND.
OHIO
KENTUCKY
TENN.
MISS.
ALA.
GEORGIA
FLORIDA
S.CAR.
N.CAR.
VA.
W.VA.
PENN.
N.Y.
MAINE

FIG. 1.

therefore, to supply storage for a four months' consumption of wheat and flour.

The most feasible means for relief to the Southeastern States is by a more extended use of the rivers of the Mississippi Valley. Wheat and flour can be collected at the important grain and flour centers such as St. Paul, Kansas City, and St. Louis, floated down the Mississippi and Missouri Rivers, and up the Ohio, Tennessee, Cumberland, Warrior, and other streams, to points of consumption. The Federal Government maintains a boat line between New Orleans, La., and St. Louis which now quotes rates 20% below the rail rates. With the new equipment in use on this line during this season, it is reported that a very handsome profit is being earned at these rates. Great improvement can be realized by the construction of adequate terminals and transshipping facilities at various points along the river. Without doubt, if these streams are properly improved and facilities for handling commodities provided equal to that found on the railroads, water rates can be maintained at as low as one-half the rail rates, and probably much below these figures when quantity shipments are made.

Several schemes have been advanced for the improvement of shipping facilities for the export trade. The destination of export wheat is shown on the wheat map, Fig. 1. It will be noted that the great bulk of the export wheat is consumed in European countries. The North Atlantic cities, including Montreal, have an advantage over New Orleans in distance to London and Continental European ports of approximately 1 500 miles. Montreal has a slight advantage over New York in distance to Liverpool, but owing to the more dangerous route down the St. Lawrence River from Montreal, and to the further fact that Montreal is only a seven months' port, New York has the advantage to all European ports. To compensate for the greater distance to Europe, the rates which shippers are now charging from Gulf ports, are from 1½ to 3 cents per bushel more than from North Atlantic points, or an average of about 2 cents per bushel. The export rail rate from St. Paul, Omaha, Kansas City, and St. Louis, through the Gulf ports, is from 5 to 7 cents per bushel less than through the North Atlantic ports. The net result, therefore, is that the Gulf ports have an advantage of from 3 to 5 cents per bushel in shipments to Europe. This advantage is reflected in the increased use of the Gulf ports. The advocates of the St. Lawrence Ship Canal argue that this advantage can be more than overcome if ocean liners can reach Chicago and Duluth. It has been shown previously that all the north central surplus is required for domestic deficiency in the East Central States. This would leave as the only surplus that from such States as Nebraska and Kansas, to be sent by water from Chicago to European points. The local rail rate from Omaha and Kansas City to Chicago is 16½ cents per bushel. After it reaches Chicago, the wheat is still nearly 1 500 miles from tide-water and more than that distance when measured in time of navigation through open water. In order to reach tide-water, it must pass through the new Welland Canal and through the improved St. Lawrence River, passing possibly sixteen locks in all before it reaches Montreal. With a normal rate of 2 cents per bushel from Chicago to Buffalo, it is probable that a rate of at least 3½ cents

a bushel would be required to deliver the wheat from Chicago to Montreal or, say, 20 cents per bushel from Omaha and Kansas City to the seaboard. This would give an apparent advantage over the Gulf ports of a total of 5 cents per bushel—2 cents in ocean haul and 3 cents in rail haul from Omaha and Kansas City to the Gulf ports. This advantage is apparent only. Wheat is not raised in Omaha and Kansas City, but at a great distance westward from these points, which distance does not add materially to the distance to Gulf ports, whereas, it must be added to any movement east through Chicago. Therefore, even with the construction of the St. Lawrence Ship Canal, it is probable that Nebraska and Kansas wheat would find its way south through Galveston and would not all be collected in Omaha and Kansas City, but at such points as Lincoln, Nebr., Topeka, Kans., Wichita, Kans., and other centers in Central and Western Nebraska and Kansas. It must be remembered that distance from Chicago to Continental Europe by water, *via* the St. Lawrence River, is approximately the same as from the Gulf ports to Continental Europe, and the time of passage through the northern route is greater, with more hazards to navigation. Moreover, the northern route is only a seven months' channel, whereas, the southern route is open throughout the year. There is, therefore, a distinct advantage in shipping export grain *via* the southern route.

In order to lower the cost of haul on export grain from Nebraska, Kansas, and Oklahoma, we must either improve rail facilities or make the Mississippi and Missouri Rivers navigable for fleets of large tonnage. There appears to be little chance of lowering the rail rate, although there is a strenuous effort being made now by agricultural interests to that end. The railroad managers are now arguing that any reduction in rates on wheat must be compensated for by additional rates on other commodities. The Rock Island Railroad, alone, estimates that if the rail rate on wheat alone is lowered 20%, the loss to that road will be $5 000 000 annually. It will be seen, therefore, that any reduction in the rail rate is simply a shifting of the burden from one commodity to another. Wheat can be floated down the Missouri and Mississippi Rivers at rates far below the existing rail rates and, with a proper development of these streams, a rate at least as low as one-half the present rail rate can be maintained with profit. This would show a saving in cost of haul from Kansas City of approximately 12 cents per bushel. In other words, a water rate could probably be maintained as low as the pre-war rail rate and, with a return to normal conditions, this rate could be still further reduced. There is an increasing demand for mill products for feed to live stock grown in the Central West. The rates should be regulated so as to encourage the grinding of wheat at Omaha, Kansas City, and St. Louis, retaining the mill product for stock purposes and shipping the finished product—flour—abroad.

Viewing the situation as shown from the wheat map (Fig. 1), there does not seem to be any excuse for any export wheat from the Central West crossing the Mississippi River. The rivers should be improved so as to accommodate craft of large tonnage and shallow draft, and terminals, elevators, and flour mills should be developed along the streams to collect and grind this wheat and float it down to New Orleans.

In addition to the saving in cost of haul, the much congested Port of New York would be relieved. The New York foreign commerce is now approximately 50% of the total foreign commerce of the United States. The congestion is so great as to add materially to the cost of haul. In the natural growth of the country, this congestion will increase, but no matter how great the facilities for added piers and terminals at the port, the collection of this great tonnage is bound to cause congestion, delays in transportation, and added interest on goods in transit.

The conclusions reached from this argument are:

1.—The producers and consumers of the United States have an added burden of $100 000 000 annually on transportation of wheat due to increased rail rates since 1914.

2.—The main economic route and destination of the surplus wheat of the North Central States is *via* the Great Lakes to Buffalo, thence by rail and water to the North Atlantic States.

3.—The South Atlantic and Gulf States, where deficiency occurs, should be supplied from the Central Mississippi Valley States *via* rail and water.

4.—European, African, South American, and Central American export wheat should come from the Central and South Mississippi Valley States and should be shipped *via* rail and water to Gulf ports.

5.—Additional milling facilities should be provided at Omaha, Kansas City, and St. Louis.

6.—The streams of the Mississippi Valley should be improved to afford an adequate navigable channel for the collection and distribution of wheat and other commodities.

7.—Modern terminals along these streams should be provided both for collection and distribution.

8.—Much may be said in favor of the proposed St. Lawrence Ship Canal, but it does not seem that its completion will materially benefit the surplus wheat raisers of America.

The St. Lawrence Ship Canal has many things to commend it, and probably it should be constructed for its general benefit to commerce and for the power that may be developed along its course; but relief to the farmers must be reached through the Southern route.

NELSON P. LEWIS,* M. AM. SOC. C. E.—The speaker has heard with great interest to what has been said on this subject, and the freedom and frankness of the discussion appeals to him. It is well for Americans to know what their Canadian neighbors are doing, and it is well to hear a protest against over-centralization and a plea for the smaller ports.

Some hysterical complaints have been heard that the Port of New York is losing its prestige by reason of the fact that the percentage of the foreign commerce of the United States, handled through the Port of New York, has greatly decreased during the last generation or two. It is quite natural that this should be so in view of the development of other parts of the country with seaports located so as to avoid excessive hauls by rail. Had not new com-

* New York City.

mercial ports been developed and had New York's percentage of the country's total commerce been maintained, the congestion due to over-centralization would have been unwholesome and unfortunate. New York Harbor has such great natural advantages that if the port is intelligently organized and adequate facilities are provided it will retain all the commerce to which it is entitled, and it need not worry about the development of other ports, whether large or small. There is needed in New York a broader conception of the whole problem and an avoidance of provincialism.

T. HOWARD BARNES,* M. AM. SOC. C. E.—One of the special topics of this discussion, namely, the relation of warehouses to port development, is timely and should be taken up more at length.

The relation is so intimate and the people are so accustomed to having a warehouse of some capacity connected with the quays and piers that they do not stop to analyze the relation of a part to the whole. This relation has been alluded to by several of the speakers, one of whom noted briefly the economy obtained where ample warehouses were so provided. The remarkable economy per linear foot of pier in certain cities has also been mentioned without a full analysis of the reasons therefor. The question is, to what extent is such economy attributable to warehousing facilities on or at the pier?

The American system, or it might be called, the primitive system, of building piers is the putting up of a structure for the quick landing of materials with a minimum of concern, at the time, beyond that of getting such materials unloaded and transferred to the shore. Even now such practice may be noted in less developed regions, and it cannot be wholly condemned in view of the uncertainty of what the future will demand. Many times, however, these developments have grown up "Topsy-like" without sufficient regard for future plans, and, thus, the opportunity for making efficient use of the original structure is forfeited.

Perhaps, the best examples of this practice are the old piers lining the North and East Rivers in New York City. At the time the earlier of these were built, there was no great reason for a storage shed more than one story in height to receive the small cargo. Congestion on Manhattan Island was not the serious question it is to-day. Trucks could come and go without the present-day delays, and near-by warehouses for storage were obtainable. To-day, all this is changed, and the speaker believes that it is an economic crime to permit the erection of piers having warehouses less than three or four stories in height.

An example of the efficiency obtained by providing ample wharf shed was illustrated a few years ago in the extension of a pier in a tropical port, which was under the speaker's charge. A large part of the imports for the interior were brought over this pier. Such imports had always been discharged on to cars, switched into the Custom House on shore, and there unloaded and sorted. The new construction provided for ample floor space on the pier, the roofed area being made level with the platform of the cars. The shed structure was provided with doors at all points by which the cargo could be

* New York City.

received as fast as the ship's sling could handle it. The time of discharging was cut substantially in two, likewise the expense of handling on shore, resulting not only in economy of handling cost, but in a very considerable saving in breakage.

All this argues in favor of having a clear space within which to receive the ship's cargo, and it must be remembered that a proper ship's tackle with plenty of winches is an effective way of discharging cargo. It must be borne in mind also that delays in discharging have been due more to a lack of opportunity to keep the cargo moving after it has been lifted out than from the fact that the ship's sling is inadequate, and that overhead cranes or other apparatus might be the remedy.

Once the cargo is landed on shore, the flow must be provided for in accordance with the demands, and it is at this stage that cranes and gantries excel. In the crowded situation in New York City, the possibility of warehousing the lighter portion, at least, in the upper stories, the convenience which this storage means to the importer, and the advantage which it affords the ship owners in accumulating cargo for export purposes, all argue most strongly for provision for that "room at the top" which the multiple-story form of construction secures.

It is to be trusted that, in his revised discussion, Mr. Long may go somewhat further into the analysis of the rate of discharge and the attendant expense as between piers provided with ample warehouses and those in which such provision is limited.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

THOMAS CURTIS CLARKE, M. Am. Soc. C. E.*

DIED MAY 25TH, 1921.

By the death on May 25th, 1921, of Thomas Curtis Clarke, the Engineer Officers' Reserve Corps lost an officer of the highest standard; the country lost a citizen who had given completely of his services as an engineer in peace and war and who was an advocate and real worker in the cause of National defense; and the Society lost a member who, in his specialty of metallurgical research, can ill be spared to the Profession.

Thomas Curtis Clarke was born in Philadelphia, Pa., on December 11th, 1873, his father having been the late Thomas Curtis Clarke, Past-President, Am. Soc. C. E.

The son of one of the most distinguished engineers of his time, Mr. Clarke, like his brothers, was educated in the Engineering Profession, having taken a special course in Metallurgy at the Massachusetts Institute of Technology. After leaving the Institute, he became for a time a Chemist at the furnaces of the Union and South Works of the Illinois Steel Company, but returned to the Institute to complete some special studies. For a few years after leaving the Massachusetts Institute of Technology, in 1893, Mr. Clarke served as an Inspector and Assistant Engineer on the construction of the Third Avenue and Willis Avenue Bridges across the Harlem River in the City of New York. Between 1897 and 1902, he was engaged in business in which, however, his engineering education was used. In 1902 and 1903, he was Treasurer of the Imboden Coke and Embree Iron Company, and had engineering charge of the construction of the furnaces of the Embree Iron Company.

In 1904, Mr. Clarke was made Treasurer and Assistant General Manager of the Astoria Steel Company. Later, he was asked to take charge of the construction and installation of the Illinois Steel Company's coal washers at Danville, Ill., a position in which his unbounded energy and experience was given full play. During 1905, he was engaged as Assistant Superintendent of the Lackawanna Iron and Steel Company, at Lebanon, Pa., in charge of five blast furnaces and by-product coke-oven plants. It was during this period, that he became interested in the specialty of coal and coke by-products, which specialty led him into much foreign travel, in investigations of by-product coal and coke plants in Germany, France, and other Continental countries.

In 1912, Mr. Clarke became interested and took charge of the development of the Niagara Company, comprising a by-product coke plant, at Buffalo, N. Y. This plant was being built under German plans and patents, but was stopped, however, at the outbreak of the World War in August, 1914, as it was being constructed largely with German capital.

* Memoir prepared by F. A. Molitor, M. Am. Soc. C. E.

Mr. Clarke had become interested in military affairs as early as 1895 when he joined the National Guard of the State of New York and was an active student of military affairs and operations.

In 1915 and 1916, foreseeing with a military eye, and aided by his experience in Germany and his acquaintance with the German people, that it was inevitable that the United States would become an active participant in the World War, he devoted his energies and time to the preparedness movement. In 1916, he assisted in the organization and training of engineer battalions in New York City, and was commissioned a Captain in the Engineer Officers' Reserve Corps when it was formed through a provision of the National Defense Act of 1916.

On the entry of the United States into the war, Capt. Clarke, with many other engineers of New York City, was ordered to the First Officers' Training Camp at Fort Oglethorpe, Ga., in May, 1917. On his graduation from this camp in August of that year, he was named as Adjutant of the 104th Engineers, a regiment organized from this training camp. Later, he was promoted to be Lieutenant-Colonel and assigned to the 110th Engineers which was the Divisional Engineer Regiment of the 35th Division. This regiment went overseas with the Division in the spring of 1918. When Col. Cheney was promoted to the rank of Brigadier General, Lt.-Col. Clarke succeeded to the command of the regiment with the rank of Colonel from August 7th, 1918. Col. Clarke commanded this regiment in the Meuse-Argonne Offensive. The regiment performed the usual work of an engineer regiment attached to an active combatant division and, in addition, it was called as infantry into the lines where it took over a sector of the front line in the active offensive then in progress.

Col. Clarke occupied as his post of command a German "pill-box", the German artillery directing their fire thereon, making it necessary for him and his immediate staff to change their post of command during the height of a severe artillery fire. The regiment acquitted itself with great credit with a loss of more than three hundred men in this emergency, two officers and seven enlisted men having received the Distinguished Service Cross for their work. The regiment was cited for its service, and Maj.-Gen. W. C. Langfitt, U. S. A., Chief Engineer, American Expeditionary Forces, commended it, as follows:

"Before issuance of definite orders for your regiment to return to the States, it is my desire that the command be advised that they have met the conditions imposed by the conflict just concluded in a most satisfactory manner.

"The construction of field fortifications and bridges across the Somme, the front-line construction, tunneling, and road work in the Vosges were notably well done. The excellent record made in front-line service is a matter of pride to the Chief Engineer as it should be to each soldier in your regiment.

"I desire that you and your command know that the services rendered were highly satisfactory and deserve commendation."

Early in Col. Clarke's service with the American Expeditionary Force, and before the Argonne Offensive, he was decorated with the Croix de Guerre

by the French Government as a result of his voluntary participation in a raid which broke through the German lines.

After the Armistice, Col. Clarke was relieved from duty with the 110th Engineers, and was appointed Acting Deputy Director of the Army Transport Service, at Tours, France, serving as such until March 31st, 1919, when at his own request he was ordered home and mustered out of the service. His interest, however, in future preparedness, and in the Engineer Officers' Reserve Corps, prompted him to take a commission as Colonel in that Corps, the rank which he held at the time of his lamentable death.

After being honorably discharged from the service, Col. Clarke became Vice-President of the International Coal Products Company, in direct and immediate charge of the engineering work. He held this position until his illness required him to relinquish it. An operation for an intestinal disorder resulted in pneumonia which caused his sudden death, his fatal illness having been caused undoubtedly by the fact that he had been gassed repeatedly while in service and when pneumonia developed, medical skill could not save him.

Col. Clarke's genial personality, wit, and unfailing patience and good nature endeared him to his hosts of friends both in and out of the Profession and they mourn his loss.

He was married on July 21st, 1897, to Elizabeth I. Knox who, with a daughter, survives him. He is also survived by two brothers, Mr. E. A. S. Clarke, for many years President of the Lackawanna Steel Company and now President of the Consolidated Steel Corporation, and Lt.-Col. Herman Clarke, of London, England.

Col. Clarke was a member of the Metropolitan Club and the Army and Navy Club of New York City, and the Rumson Country Club. He was also a member of the American Institute of Consulting Engineers, Society of Chemical Industry, Society of American Military Engineers, and the Military Order of the World War.

Col. Clarke was elected a Member of the American Society of Civil Engineers on May 4th, 1909.

WILLIAM HARPER ROBINSON, M. Am. Soc. C. E.*

Died December 29th, 1920.

William Harper Robinson, the son of Judge Robert Robinson and Luisa Harper Robinson, was born on February 10th, 1868, at Sacramento, Cal. He was educated in the public schools of San Francisco, Cal., and after having been graduated from the Boys' High School of that city, attended Heald's Business College, the Mechanics' Institute of San Francisco, and the Van der Naillen School of Engineering.

In 1887, he entered the employ of the Southern Pacific Railroad Company, for which Company he worked continuously, as Draftsman, Leveler, Transitman, and Assistant Engineer, for a period of ten years. During that time, Mr. Robinson was Draftsman on the following Southern Pacific surveys:

* Memoir prepared by L. Fred Patstone, M. Am. Soc. C. E.

From Santa Ana to Oceanside, Burbank to Chatsworth Park, Long Beach Junction to Long Beach, San Pedro to Point Fermin, Ventura to Nordhoff, Montalvo to Simi Pass, Oakdale to Merced, and on double-track construction from Oakland to Port Costa, all in California. During 1889 he served as Draftsman in the General Office of the Company in San Francisco.

In 1890, Mr. Robinson was employed as Leveler on the following Southern Pacific surveys: From Riverside to San Bernardino, Colton to Redlands, and Avon to Walnut Creek. During the same year, he was Transitman on the line from Santa Monica to Soldiers' Home, and on various main-line changes in California, Arizona, and New Mexico.

In 1891, he was engaged as Transitman and Topographer for the Ferrocaril Occidental de Guatemala, on the line between San Felipe and Quezaltenango.

The year 1892 found Mr. Robinson again with the Southern Pacific Railroad as Draftsman in the General Office in San Francisco. In 1893, he was engaged as Transitman on surveys for the Company from Gaviota Landing to Lompoc Landing and from San Luis Obispo to Lompoc Landing, and, in 1894, he was employed as Assistant Engineer on line changes on the Park and Ocean Railroad, on relocation from San Bruno to San Francisco, and on line changes between San Miguel and Santa Margarita. In 1895, he was again in the General Office in San Francisco. In 1896, he served as Assistant Engineer on the construction of standard road between Riverside and Colton and on water development work on the Hope Ranch in Santa Barbara County, California.

In 1897 and 1898 Mr. Robinson was engaged in the private practice of his profession, largely on surveys, land irrigation, and water development. In 1899-1900, he was employed by the Redlands Electric Light and Power Company on the location and construction of the power plants in Mill Creek Canyon, San Bernardino County, California.

In 1900, he went to the Philippine Islands where he was engaged as Leveler and Surveyor on the Benguet Road, and in 1901, he became Assistant to the Superintending Engineer of the Army Transport Service at Manila. In 1902, he was appointed as Supervisor of the Province of Misamis and, in 1903, was transferred to the post of Assistant Engineer to the Philippine Commission. Soon after this, he was transferred to the Coast Guard Service as Principal Assistant Engineer in the Division of Lighthouse Construction. Mr. Robinson resigned this position, in 1905, to accept that of Superintendent of Buildings for the Philippine Railroad, which position he held for three years, during which time he designed and built all the depots and terminals of that road.

In the fall of 1908, he again entered the Philippine Government Service as Division Road Engineer in the Bureau of Public Works, and from this time, his promotion in that service was rapid. He passed through the positions of Division Engineer, Chief Division Engineer, Principal Assistant Engineer, and Acting Assistant Director, until on June 6th, 1910, he was appointed City Engineer of Manila and Head of the Department of Engineering and Public Works, which position he held until August 1st, 1916.

During this time, Mr. Robinson was a big factor in developing the City Plan for the future improvement of Manila and in establishing and improving many of its public works. As *ex-officio* member of the Municipal Board of Manila, he was able to give engineering advice and guidance in many of the great problems incident to the growth and well-being of the city.

Owing to ill-health, Mr. Robinson was forced to resign in August, 1916, and return to the United States. He purchased a ranch in Marin County, California, where he continued to reside until his death which occurred on December 29th, 1920. He is survived by his wife, Emma Upchurch Robinson, to whom he was married in San Diego, Cal., on March 4th, 1896.

After Mr. Robinson's return to the United States, ill-health prevented him from engaging in active work, although he retained a keen interest in his chosen profession. At the time of his death, he was in charge of the street and paving improvements in Dixon, Cal.

Mr. Robinson was an exceptionally fine man with whom to work, and all engineers who came in contact with him appreciated his broad-mindedness and his spirit of co-operation. He was the type of man who is progressive and is not afraid to shoulder responsibility. Throughout the Philippines, and especially in Manila, there are many public works which stand as monuments to his memory and as mute testimonials to a long and successful career in the engineering work of the United States Government in the Philippines.

Mr. Robinson was elected a Member of the American Society of Civil Engineers on March 1st, 1910.

JOHN WILSON, M. Am. Soc. C. E.*

Died June 28th, 1921.

John Wilson was born in Wisbeach, Cambridgeshire, England, on January 23d, 1841. He was graduated from Cooper's Hill Civil Engineering College, in the suburbs of London, in 1865, and the following year was sent by the English Government to India as an Engineer in the Public Works Department.

Later, Mr. Wilson was appointed Assistant Engineer on the Ganges Canal, after which he was transferred to Bengal, India, as Engineer on the construction of the Sone Canal. In 1876, famine broke out in Bengal, and he was sent to the Provincial Branch for famine relief work and was assigned to the District of Dinagepore in the low swampy grounds of the Ganges Valley. There he served until the famine was over and gained the title of Major, by which he was ever afterward known.

In 1882, Major Wilson came to the United States and was engaged as Consulting Engineer on the construction of cableways in Philadelphia, Pa., Chicago, Ill., and Pittsburgh, Pa. In 1892, he went to Texas to take up the survey and design of a hydro-electric power plant near Waco.

In 1894, Major Wilson was appointed Engineer for the Estate of the Corralitos Ranch and Mining Company in the State of Chihuahua, Mexico. During this engagement he surveyed the ranch of a million acres and planned

* Memoir prepared by Vernon L. Sullivan, M. Am. Soc. C. E.

and designed the improvements relative thereto, which included the building of dams and the laying of miles of water pipe.

In 1899, Major Wilson was engaged by the El Paso Electric Railway Company on the construction of the electric railway system for El Paso, Tex. During 1904, 1905, and 1906, he designed and constructed the irrigation system for the Big Valley Irrigation Company, at Grand Falls, Tex., from which work he went as Chief Engineer and Manager for the Barstow Irrigation Company, at Barstow, Tex. He remained at Barstow until he was appointed a Member of the Board of Water Engineers for the State of Texas in September, 1913. He served on this Board for two terms, after which he retired from active practice and moved to California.

He was a man of strong character and of great determination, adhering strictly to his own ideals, regardless of public comment, and, in his death, the people lose a man who would have helped to make the world better.

Major Wilson was elected a Member of the American Society of Civil Engineers on March 2d, 1915.

JAMES GIBBONS BROWNE, Assoc. M. Am. Soc. C. E.*

Died April 25th, 1921.

James Gibbons Browne, the son of John T. and Mollie Bergin Browne, was born on December 16th, 1886, at Houston, Tex. He received his preliminary education in the public schools and parochial schools at Houston, and entered the University of Texas in the fall of 1904, from which he was graduated in June, 1908, with the degree of Civil Engineer.

From 1908 to 1910, Mr. Browne was employed in various municipal improvements in the City of Houston and in some of the smaller cities of Texas, and, in 1910, he engaged in practice for himself, undertaking municipal work, sewage, pavements, bridges, and similar contracts.

Mr. Browne made rapid progress and, at the time he was stricken with what later proved to be a fatal illness, was regarded as one of the most promising young engineers in the construction field in Southern Texas. In the fall of 1915, following an attack of ptomaine poisoning, serious and complicated illness developed. In spite of the services of several specialists, and most conscientious efforts to find a cause and remedy for this malady, Mr. Browne grew steadily worse. He passed most of the year preceding his death on April 25th, 1921, in a state of coma.

Mr. Browne's grasp of the business side of construction work, and his large fund of common sense, promised for him a most successful and useful career. His kindly and generous personal qualities cannot be forgotten by those who knew him best.

On November 24th, 1910, he was married to Annie Marguerite Casperson, of Houston, Tex., who, with one child, Mary Marguerite, survives him.

Mr. Browne was elected an Associate Member of the American Society of Civil Engineers on May 6th, 1914.

* Memoir prepared by J. C. Stevenson, M. Am. Soc. C. E.

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INSTITUTED 1852

PAPERS AND DISCUSSIONS

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Vol. XLVII. NOVEMBER, 1921. No. 9.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

A REVIEW OF IMPORTANT DEVELOPMENTS IN THE SCIENCE OF CADASTRAL RESURVEYS AS EXECUTED BY THE UNITED STATES GOVERNMENT, WITH ETHICAL DISCUSSION THEREOF

By Howard Richards Farnsworth,* Assoc. M. Am. Soc. C. E.

Synopsis

Owing to the great number of interesting phases appropriate for discussion in any general review of the science of cadastral resurveys, this work has necessarily been restricted, and is confined to a study of some of the more recent and important of the technical improvements in field and office methods, as developed in the Cadastral Engineering Service of the United States General Land Office.

It must be assumed that the reader is familiar with the long established official method of execution of original public land surveys, which produces a rectangular network of 36 sections per township 6 miles square, with monuments established at intervals of ½ mile. At the present time, the original areas are nearly all subdivided and platted, and the greater part of what was once a vast undeveloped public domain has passed into private ownership. The rapid development of this area and of its latent wealth depends largely on a quiet title and undisturbed possession, which, if not secure, immediately retards the growth of the mineral, lumber, agricultural, and live-stock interests which in large measure form the basis for credit at the banks and that National wealth which is the bulwark of the country.

Until 1910, the Government executed these original surveys by entering into contract with a successful bidder, generally on a mileage basis. An unfortunate result was the frequent careless and sometimes fraudulent work, especially in the early surveys, the natural desire of the deputy surveyor being to execute his contract with the minimum of expense and danger to himself. The early surveys were often a mass of errors, sometimes with no monu-

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* Washington, D. C.

ments set at all, a fictitious plat and field record having been prepared and filed which described the true extent and character of the lands represented with only remote accuracy. Again, especially on the Western plains, Nature and the elements have gradually obliterated the corner monuments, important boundaries and whole counties being thus affected. Finally, when settlement began, a vain search for corners commenced, strife ensued, and unending lawsuits developed among the claimants and owners of the lands.

Other great surveying departments of the Government, such as the United States Coast and Geodetic Survey, have always used the most refined methods, a direct method of execution instead of contract, and have taken ample time to measure and calculate distances and angles properly. Yet, when errors are found in their work, no law intervenes to prevent their correction and the moving of the monuments. Corners of public land surveys, however, on which private rights have been based, cannot be changed. Other surveys remain as before, subject only to their scientific authors, whereas the cadastral survey becomes a creature of the Courts.

In order to satisfy the ever-increasing demand for the scientific and legal treatment of this "error", Congress has provided for the Federal execution of cadastral resurveys. For the past eleven years, this complicated and exacting work has been successfully prosecuted by the cadastral engineers of the Government, who are applying only the most approved scientific and legal methods. It is believed that every civil engineer having to do with field work, will find the discussion of the subject, as set forth in this paper, to be of interest and value.

ADMINISTRATION

Organization and Duties

The tenth anniversary of the inauguration of the direct system of surveys and resurveys by the General Land Office was reached on June 25th, 1920, on which date, in 1910, Congress discontinued the old contract system* and the work was placed under the direct supervision of the Commissioner of the General Land Office. The record of all classes of accepted surveys under the new system, for the past ten years, is estimated at 100 109 570 acres, or an average of 10 000 000 acres per annum, the high mark having been reached in 1915. The aggregate area resurveyed during the fiscal year ending June 30th, 1920, was 2 514 306 acres.†

The administration and technical direction of this stupendous work is vested in a small group of civil engineers in what is known as the Division of Surveys and Resurveys. With the advice and counsel of a Board of Law Review, these engineers act for the Commissioner in the attainment of correct execution and practical results in what is undertaken. This Division is the administrative unit which undertakes the administration, supervision, technical control, and execution of the survey of public lands together with other

* For an exposition of this system, see the paper by C. L. DuBois, Mass. Inst. of Technology *Quarterly*, Vol. XII, No. 4 (December, 1899).

† Report of the Commissioner of the General Land Office to the Secretary of the Interior, June 30th, 1920, pp. 19 and 127.

surveys involving the boundaries of Mexican and Spanish private land grants, of State and international boundaries, and resurveys authorized by law.*

Fig. 1 is a diagram of the administrative organization of the Cadastral Engineering Service of the General Land Office as of 1919, showing the relative supervisory responsibilities.

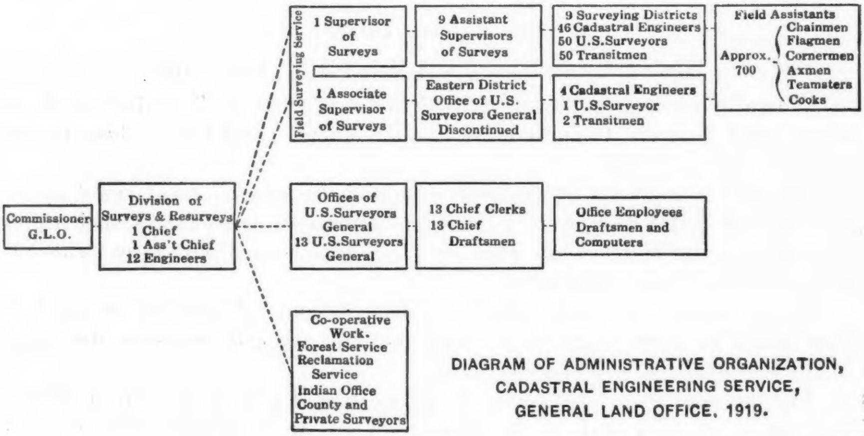

FIG. 1.

AUTHORITY OF LAW

In general, the legislation provided by Congress for the execution of cadastral resurveys under this organization may be divided along the lines of the application thereof to two general conditions, first, where title to more than 50% of the land to be resurveyed remains in the United States, in which case the Government bears the entire cost; and, second, where disposals are in excess of 50% of the total area, in which case the private holder bears his share of the expense as well as the Government, proportioned in accordance with the acreage held. This service, which has become invaluable to the Western States, has recently been made available to the Central and Eastern States where unending boundary disputes are still found to obtain, although for many years the lands have been held in private ownership.

The first general Act was approved by Congress on March 3d, 1909, and was entitled "An Act Authorizing the Necessary Resurvey of Public Lands." It reads in part as follows:

"That the Secretary of the Interior may * * * cause to be made * * * such resurveys or retracements of the surveys of public lands as * * * he may deem essential to properly mark the boundaries of the public lands remaining undisposed of: Provided, that no such resurvey or retracement shall be so executed as to impair the *bona-fide* rights or claims of any claimant, entryman, or owner of lands affected by such resurvey or retracement." (35 Stat., 845).

* Further information on this subject may be had by reference to the decision of the U. S. Supreme Court in United States *vs.* Morrison (240 U. S., 210–212 inclusive); to the 1919–21 files (Form No. 28), of the U. S. Bureau of Efficiency; and to the report dated March 12th, 1920, of the Federal Reclassification Commission.

The second general Act, entitled "An Act Authorizing the Resurvey or Retracement of Lands Heretofore Returned as Surveyed Public Lands of the United States under Certain Conditions" (40 Stat., 965), was approved September 21st, 1918. Work under the latter Act is undertaken on the application of the owners of three-fourths of the privately owned lands, or of any Court of competent jurisdiction.

Restrictions Imposed

From a review of Congressional legislation it is evident that:

Where private rights have attached to boundaries of the public lands, as duly accepted by the Commissioner of the General Land Office, these boundaries are unchangeable.

Original monuments of all classes on such boundaries must stand as the true corners which they were intended to represent, in so far as such rights are affected, irrespective of whether their actual positions agree with the record of the original field notes.

Where corners on such boundaries have become obliterated or are lost, they must, in a resurvey, be restored in their original positions, according to the best available evidence thereof.

The jurisdiction of final review in all matters of private ownership affected by a resurvey, is vested in the Courts. Thus, it is evident that, in suits involving boundary disputes, the rules of procedure laid down to guide the U. S. Cadastral Engineer in his execution of resurveys must be in harmony with the leading Court decisions, in order that, when properly applied, the Courts may accept without question the boundaries thus determined in so far as they represent the true location of a particular tract of land intended to be conveyed by a patent.

IMPROVEMENTS IN THE FIELD

In Equipment

The modern practices developed and utilized in the execution of Federal cadastral surveys are in keeping with the orderly advance in refinement and precision to be found in the methods used in all other engineering and scientific activities, when compared with the practice of years ago. A 1919 edition of the "Manual of Surveying Instructions", which embraces six of the ten chapters contemplated for the completed edition, completely revises all previous issues with additional new material. This edition has been prepared by a Board, created by the Commissioner, composed of four members of the Engineering Staff of the General Land Office, together with a field officer, which, as a body, deserves much credit for expanding and establishing the practice of a highly specialized science.

A few of the more convenient and useful forms of observation as taken from these recent "Instructions",* will be given in the pages that follow. Some of these forms are not in general use and will appeal to the field engineer.

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of the Public Lands of the United States", General Land Office, 1919, p. 23.

Alignment.

Improved Solar Transit.—The engineer's transit, as equipped with the Smith solar attachment, has been developed in the General Land Office to a state of efficiency which fully warrants the adoption of this model as a standard instrument. In improving the construction, special attention has been given to provide suitable means by which working parts may be properly adjusted to insure uniform accuracy in the results attained. No detail has been omitted, and the perfected arrangement of the working parts permits a precise and rapid adjustment in the field, in a simple manner readily understood by any competent operator and entirely free from any uncompensated or residual errors. Adequate provision has been made for a maximum protection of the delicate working parts, with due attention given to compactness and to a proper distribution of weight.

A result of special tests in this service, where a large number of every form of solar model are constantly in use, has proven the Smith solar attachment to be far superior in efficiency to all other forms. In improving the construction of this solar attachment to obtain the standard instrument, the process of evolution from the old to the new model has been gradual and tedious, each change being put to the test of months of actual field use.

In the improved construction, a regular light mountain model, full engineer's transit is used, on the east standard of which is mounted the solar attachment. The essential features of the important improvements over older models are as follows:*

1.—The solar has been mounted on an instrument having V-shaped standards, thereby adding much to the stability of the attachment.

2.—The base-plate of the solar is mounted on three foot ~~3-ft.~~ posts, thus relieving the strain due to imperfect adjustment of the older models having a four-point base.

3.—The position of the base-plate is adjustable by opposing capstan nuts on the foot posts, each with a countersunk ball washer, thereby obtaining positive adjustment altogether free from strain on the capstan nuts.

4.—The three-point base forms a right-angle triangle, with one side horizontal and one side vertical, thereby permitting adjustment in either of two directions: (*a*), One about a horizontal axis, and (*b*), one about a vertical axis, either without disturbing the other.

5.—The axis of the latitude arc is arranged so that its position may be tested with a striding level without removing the auxiliary telescope.

6.—Both the latitude arc and auxiliary telescope are hung beneath the latitude axis, thereby lowering the center of gravity of the attachment and giving much greater protection to the delicate working parts.

7.—Suitable capstan nuts have been placed at one end of the auxiliary telescope to provide for its proper adjustment, with respect to the axis of the latitude arc.

---

* A. D. Kidder, "A Description of the Smith Solar Attachment, as Recently Improved for the Surveying Service of the General Land Office", published under the direction of the Commissioner, General Land Office, 1915, p. 5.

8.—Improved interlocking devices have been placed on the latitude and declination arcs, verniers, clamps, and tangent motions.

9.—The mirror may be swung around instantly to permit direct sighting through the auxiliary telescope.

10.—Absolute freedom of motion of the various working parts each to perform its own function, and each one independently, quickly and permanently adjustable.

Maximum efficiency may be expected and obtained from the operation of the solar attachment only when the attachment itself, as well as the transit on which it is mounted, is in proper adjustment and when so adjusted efficient meridional performance may be obtained. The recent improvements have added greatly to the precision and ease with which the adjustments may be accomplished. These adjustments are described in detail in the advance sheets of the "Instructions" issued under the direction of the Commissioner, previously referred to, and are well worthy of adoption by all engineers interested in time, latitude, and azimuth determinations by the use of this model. (Fig. 2.)

Measurement.

Triangulation and Stadia.—The Federal engineer is authorized to obtain distances across water and over precipitous slopes by the use of appropriate triangulations or a properly safeguarded stadia method. In the former, care is exercised in the selection of the measured base and the adoption of the best possible geometric proportions of the sides and angles of the triangle. When it is desired to determine the value of any angle with a precision less than the least reading of the instrument, the method of repetitions is utilized. In the use of the stadia method, the wire interval or ratio is required to be determined in the field by frequent tests and under working conditions, in comparison with steel tape measurement, solving the following formula* for the value of the wire ratio when the horizontal distance is known:

$$\text{Horizontal distance} = K\ r \cos^2 V + (c + f) \cos V$$

in which

"horizontal distance" = the distance from the center of the instrument to the rod;

K = the wire ratio;

r = vertical rod reading;

c = distance from center of instrument to object glass;

f = distance from plane of cross-wires to object glass; and

V = observed vertical angle.

Long Steel Tape and Clinometer.—Where it is possible, the most approved method of measurement, however, is that involving the use of steel ribbon tapes from 2 to 8 chains in length, checked frequently against a suitable reference standard.

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* "Tables and Formulas for the Use of U. S. Surveyors and Engineers on Public Land Surveys," Commissioner of the General Land Office, Second Edition, 1913, p. 221.

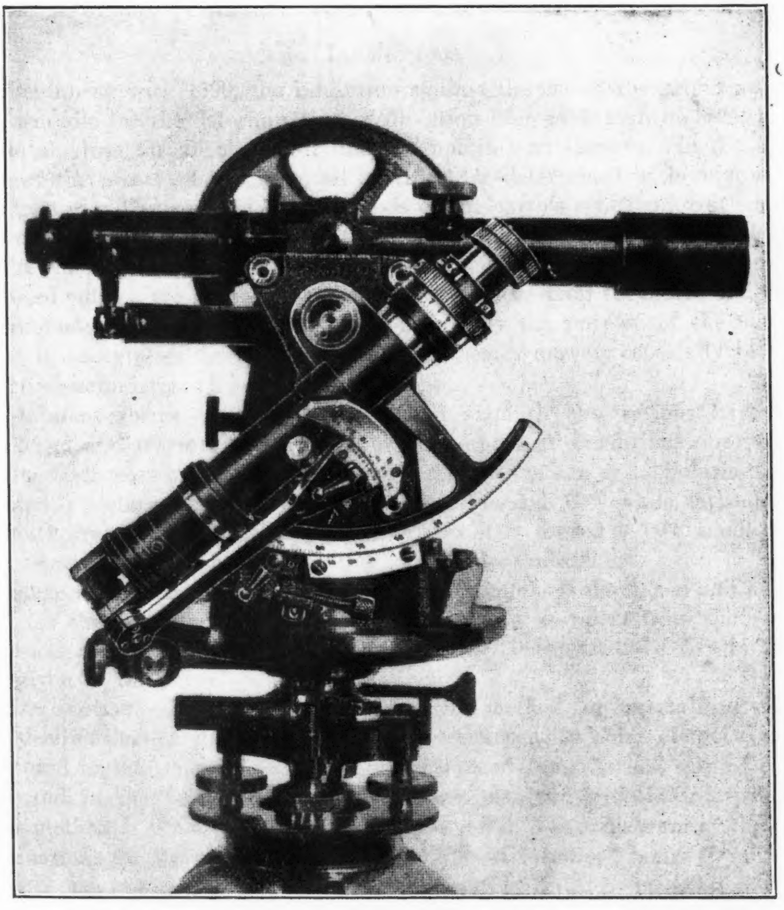


FIG. 2.—THE IMPROVED SOLAR TRANSIT.

The long tape when used is properly aligned and stretched so that the measurement may be made on the slope. One of the most rapid and reliable methods of slope measurement is obtained by a skillful use of the long steel tape. The transit is used to determine vertical angles of particularly sharp slopes, the angle of lesser slopes being determined with the clinometer. Slope distances are then reduced to true horizontal distances, and the entire operation is recorded. Rapidity in the reduction of slope measurements is obtained by the use of appropriate stadia-reduction diagrams.

In Methods.

Beginning with 1890, the inhibition against the use of the primitive magnetic needle for the determination of direction increased, and, in 1894, it was made absolute for all classes of lines on public land surveys. In the execution of the present-day survey, all bearings are determined with reference to the true meridian as defined by the axis of the earth's rotation. Such meridian determinations are required at the beginning, at necessary intervals, and at the conclusion of each work. The instruments are of the latest design and are used only on the approval of a supervising officer after they have undergone a satisfactory test in the field. It is now truly the purpose of the Government to accomplish final results in all its lines of survey, especially in those affecting boundaries of private vested rights.

Different forms of time, latitude, and azimuth observations have been developed and rearranged to facilitate work under all conditions encountered in the field, most of which are referred either to the sun or to Polaris. Time is readily determined with an error not to exceed 10″, while latitude and azimuth are readily available with an error not to exceed 1′ 00″, small errors in assumed longitude being neglected in such determinations.

Some of the more useful of the advances in methods developed and authorized for the execution of official cadastral surveys, as taken from the general instructions issued by the Commissioner of the General Land Office in 1919,* are given in the pages which follow.

Declination.—The advantages of a graphic method for ascertaining changing declinations of the sun, corrected for refraction in polar distance, are to be found in the practical elimination of errors of computation, ease of checking, and in the fact that actual values are obtained without interpolation. This method is illustrated by Figs. 3 and 4. Fig. 3 is a diagram of the sun's declinations for March 20th, 1912; Lat. 37° 30′ N.; Long. 7 hours 30 min. W.:

$$\begin{array}{lcl}
\text{Declination, Greenwich noon} = 4.30 \text{ A. M.} & = & 0^\circ\ 11'\ 14''\ \text{S.} \\
\text{Difference, 10 hours} = +\ 593'' & = & 09'\ 53''\ \text{N.} \\
\hline
\text{Declination at 2.30 P. M.} & = & 0^\circ\ 01'\ 21''\ \text{S.}
\end{array}$$

and Fig. 4 is a diagram of the sun's declinations for September 23d, 1913; Lat. 47° 30′ N.; Long. 6 hours 18 min. W.:

$$\begin{array}{lcl}
\text{Declination, Greenwich noon} = 5.42 \text{ A. M.} & = & 0^\circ\ 03'\ 55''\ \text{N.} \\
\text{Difference, 10 hours} = -\ 585'' & = & 9'\ 45''\ \text{S.} \\
\hline
\text{Declination at 3.42 P. M.} & = & 0^\circ\ 05'\ 50''\ \text{S.}
\end{array}$$

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of Public Lands of the United States", 1919, pp. 23, 48–49.

In Figs. 3 and 4, the horizontal lines represent each hour of the day, and the vertical lines represent 1-min. intervals in declination, the latter being numbered to suit the range of the sun's declination for the date. Two points

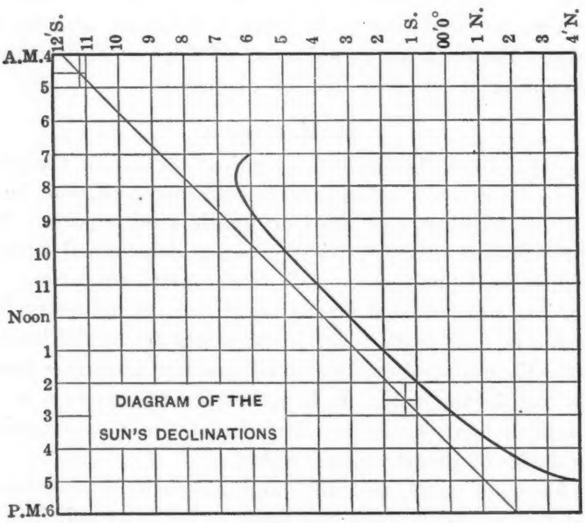


FIG. 3.

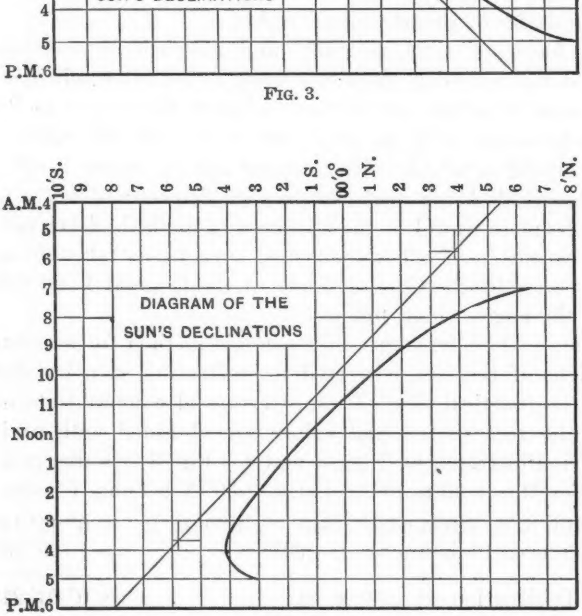


FIG. 4.

are marked on the diagrams to agree with the true declination of the sun. The first point is marked with the argument of declination agreeing with that given in the Ephemeris* for Greenwich apparent noon, that of time agreeing

* "Ephemeris of the Sun and Polaris, and Tables of Azimuths and Altitudes of Polaris", Commr. of the General Land Office, pub. annually since 1910.

with the local apparent time corresponding to Greenwich noon. The second point is marked to agree with the proper declination and time, 10 hours later. A straight line connecting the two points will then form the locus of all points which indicate the sun's true declinations for the date, for local apparent time. The proper refractions in polar distance are then scaled from the straight line toward the north for each tabulated value, morning and afternoon, as given in any correct table of values of mean refractions in polar distance, appropriate for the latitude of observation and declination of the sun. The locus of these points as plotted is a smooth curve representing the sun's declinations corrected for refraction in polar distance. The scale used for plotting the refractions must equal the scale of the intervals of 1 min. in declination, and the refractions must be laid off parallel to the horizontal lines and not normal to the line of true declination. A reading from the curve at the point corresponding to the time of observation will then give the proper declination for use with the solar attachment. For use in the reduction of altitude observations, the true values of the sun's declination will be taken from the straight line.

Time.—It is necessary for the cadastral engineer to know the exact apparent time of all his observations taken on the sun and the true local mean time of all observations taken on Polaris. When comparison with standard telegraph time is impracticable, direct determinations become necessary, and the method used will be either an altitude observation of the sun for apparent time, meridian observation of the sun for apparent noon, meridian observation of a star of appropriate magnitude, and declination for local sidereal time, or time secured directly by use of the solar attachment. Determinations by the latter method, however, are restricted to use in observations on Polaris at either elongation, and, of course, are not sufficiently accurate for observations by the hour-angle method. The meridian observation of the sun is by far the most convenient reliable method of time observation. The observing programme for this method is as follows: Determine the meridian by the best means at hand and compute the altitude setting for the sun; level the transit, place the instrument in the meridian, and elevate the telescope to the altitude of the sun's center; note the watch time of the sun's west limb tangent to the vertical wire; and note the watch time of the sun's east limb tangent to the vertical wire.

Take the mean of the readings for the watch time of apparent noon from which to compute the watch error, local mean time. If the observation fails for either limb, the reduction to the sun's center is accomplished by adding or subtracting 68 sec. A refinement in this amount is had by reference to the Ephemeris* for the time of the sun's semi-diameter passing the meridian for the date of observation. The setting for the approximate altitude of the sun's center is:†

$$V \neq 90° - \phi \pm \delta$$

* "Ephemeris of the Sun and Polaris, and Tables of Azimuths and Altitudes of Polaris", Commr. of General Land Office, pub. annually since 1910.

† "Advance Sheets of a Revision of the Manual of Instructions for the Survey of Public Lands of the United States", 1919, p. 65.

The field record appears somewhat as follows:* August 14th, 1909, in latitude 37° 16′ N.; longitude 102° 16′ W.:

$$
\begin{array}{rl}
\text{Setting:} & 90^\circ\ 00' \\
\phi \neq (-) & 37^\circ\ 16'\ \text{N.} \\
\delta \neq (+) & 14^\circ\ 25'\ \text{N.} \\
\hline
V \neq & 67^\circ\ 09'
\end{array}
$$

	h. m. s.
°+ Watch time of transit, W. limb =	11 59 22
+° Watch time of transit, E. limb =	12 01 32
Watch time apparent noon =	12 00 27

	h. m. s.
Apparent noon =	12 00 00
Equation of time =	+ 4 33
Local mean time of apparent noon =	12 04 33
Watch slow of local mean time =	0 4 06

In this and in other observations hereinafter given, the following analytical notation is used, to follow that used in the Commissioner's "Instructions":

Let $\neq$ = an inequality which approaches equality;

V = observed vertical angle; in altitude observations on the sun, the mean observed vertical angle to the sun's center.

h = true vertical angle to the sun's center, or to Polaris, in altitude observations, after correction for refraction: $h = V -$ refraction in zenith distance. In altitude observations on the sun, a refinement is had by adding the value of the sun's parallax $= 8.9'' \cos V$, opposite in effect to refraction.

ζ = true zenith distance of the sun's center $= 90^\circ - h$.

ϕ = latitude of the station of observation.

λ = longitude of the station.

δ = declination of the sun or Polaris.

Latitude.—A definite knowledge of the true latitude is very important in the use of solar instruments. No lack of reasonable precision is allowed in an accepted latitude and for this reason a considerable series of latitude determinations is taken on every large work, which, on comparison, will produce a satisfactory mean. Altitude observations of Polaris at the upper or lower culmination and meridian altitude observations of the sun for latitude are the methods used. The latter is used most extensively, a series of observations being taken on successive days. It is easily executed and appears as follows: Level the transit with the telescope in the meridian elevated to the sun's approximate altitude at noon; observe the altitude of the sun's lower limb with the sun slightly east of the meridian; reverse the transit; observe the altitude of the sun's upper limb with the sun slightly west of the meridian; and take the mean observed vertical angle for the altitude of the sun's center at apparent noon.

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of Public Lands of the United States", 1919, p. 65.

The field record is as follows:* October 5th, 1909, in approximate latitude 37° 20′ N.; longitude 102° 04′ W.:

$$
\begin{array}{lrl}
\text{Setting:} & & 90^\circ\ 00' \\
 & \phi \neq (-) & 37^\circ\ 20'\ \text{N.} \\
 & \delta \neq (-) & 4^\circ\ 42'\ \text{S.} \\
\hline
 & V \neq & 47^\circ\ 58' \\
\text{Lower limb} \ldots\ldots & & 47^\circ\ 42' \\
\text{Upper limb} \ldots\ldots & & 48^\circ\ 14'
\end{array}
$$

$$
\begin{array}{lrl}
{}^\circ{+}\ \text{Observed altitude, lower limb, direct} & = & 47^\circ\ 43'\ 00'' \\
+_\circ\ \text{Observed altitude, upper limb, reversed} & = & 48^\circ\ 16'\ 30'' \\
\hline
\text{Mean observed altitude,} & V = & 47^\circ\ 59'\ 45'' \\
\text{Refraction} - & & 0'\ 52'' \\
\text{Parallax} + & & 0'\ 06'' \\
\hline
h & = & 47^\circ\ 58'\ 59'' \\
\delta & = & 4^\circ\ 41'\ 42''\ \text{S.} \\
\phi = 37^\circ\ 19.3'\ \text{N.} = 90^\circ - \delta - h & = & 37^\circ\ 19'\ 19'' \\
\hline
 & & 90^\circ\ 00'\ 00''
\end{array}
$$

Meridian observations of the sun for time and latitude are conveniently combined, by observing simultaneously the sun's lower and west limbs, recording the watch time and the vertical angle, and reversing the transit in the interval of about 2 min., and then observing simultaneously the sun's upper and east limbs. Settings for approximate altitudes of the sun's lower and upper limbs are, respectively:

$$V \neq 90^\circ - \phi \pm \delta \mp 16'$$

Azimuth.—There are a variety of methods and number of heavenly bodies from which to select for determinations of azimuth. A solar transit properly adjusted to the true meridian will serve best on line work in timbered country. In open country, however, it will often be found practicable to carry forward a transit line, always referred to the true meridian by deflection angles. The writer has frequently completed the resurvey of entire townships by deflecting the necessary angles from an initially determined stellar meridian, and when checking the last line run, against the true meridian, has intersected a spade handle set as a reference point. This may readily be accomplished by the careful handling of an instrument which is kept in perfect adjustment.

Methods of observation on Polaris and on the sun are unquestionably the most desirable for determinations of azimuth for the use of the cadastral engineer. Observations on Polaris at its eastern or western elongation, at any hour-angle, most conveniently taken at sunset or sunrise, and direct altitude observations and equal altitude absorvations of the sun are among those used. The latter method, however, has little adaptability to line work, but by virtue of the need only of approximations of time and latitude, the method possesses a certain usefulness in camp.

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of the Public Lands of the United States", 1919, p. 67.

By far the most convenient and accurate azimuth determination is by observation of Polaris at any hour-angle, preferably during the daylight hours, thereby eliminating the necessity for artificial illumination and the personal equations of various assistants. However, with the direct rays of the sun above the horizon at sunset or sunrise, a close approximation of the position of Polaris is necessary before it may be found. The hour-angle, t, and azimuth, A, are ascertained in order to locate Polaris in azimuth. Its position in altitude is determined by the following approximation,* the positive sign being used for hour-angles of less than 6 hours and the negative sign for hour-angles exceeding 6 hours:

$$V \neq \text{Latitude} \pm (\text{Polaris distance of star} \times \cos t) \neq \phi \pm 70' \cos t.$$

A computation* of the position of Polaris at sunset, May 6th, 1911, at a station in latitude 47° 20′ N., and longitude 102° 40′ W., appears somewhat as follows: The declination of the sun is found from the Ephemeris† to be 16° 18′ N., and the apparent time of sunset is found to be 7 hours 15 min., P. M., by reference to the Standard Field Tables.‡ Then the

Assumed time of observation, May 6th, 1911		=	h. m. 7 15 P. M.
Gr. U. C. of Polaris, May 6th =	h. m. 10 33.5 A. M.	+	12
Reduced to Long. 102° 40′ W.	— 1.1	=	10 32.4 A. M.
Assumed hour-angle of Polaris west of the meridian..		=	8 42 6
Hour-angle, angular measure..................		=	130° 39′
Azimuth of Polaris, ≠ sin	130° 39′		
× (azimuth at W. E.)		≠	1° 17′ W.
Latitude of station =	47° 20′		
70′ cos t = 70 cos 130° 39′ =	46 (—)		
$V \neq$	46° 34′		

In order to find Polaris an approximate meridian must be available as a reference from which to set off a horizontal angle of 1°17′ W., and a vertical angle of 46°34′.

In the twilight hours, after the passing of the direct rays of the sun below the horizon, only a rough approximation of the position of Polaris is necessary in order that it may be found, and there still remains enough daylight for clear identification of the reference mark and reading of the verniers. The writer has often determined the necessary co-ordinates of position roughly from a glance at a Hammet's planisphere set in position for the time and date. Although this method may not at first bring Polaris within the field of the telescope, by small quick movements of the telescope in departure or in latitude, the flicker of the star's light across the field will indicate its presence where a steady, direct sight will not. Especially is this the case when the pre-deter-

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of the Public Lands of the United States", 1919, p. 100.

† "Ephemeris of the Sun and Polaris, and Tables of Azimuths and Altitudes of Polaris", Commr. of General Land Office, pub. annually since 1910.

‡ "Tables and Formulas for the Use of U. S. Surveyors and Engineers on Public Land Surveys," ed. 2, Commr. of General Land Office, 1913.

mined sidereal focus of the telescope is found not to be true for use on the star. In this observation, a distant object is greatly to be preferred as a reference mark for reading horizontal angles, in order that this focus, when corrected for the star, will not have to be changed when sighting on the mark.

With a number of equations at the disposal of the engineer to suit his convenience, a very efficient alternative for azimuth observations on Polaris is found in direct altitude observations on the sun. These observations are useful in that they can be taken on the line of survey during the day and used for the required tests of the solar attachment, which during appropriate hours for solar work is expected always to come within 1′30″ of the true meridian, before approval.

Under working conditions any line determined with the solar attachment may be used for reference purposes from which to obtain the necessary data for computing the true bearing thereof. In order to guard against error, a series of three altitude observations on the sun, each with the telescope in direct and reversed positions are required. These are readily obtained in 10 or 12 min., while the reductions may be made in the evening. When using this method of observations a full vertical circle, a colored glass shade in the dust shutter of the eyepiece, and a prismatic eyepiece are essential to rapidity of performance and accurate results. The standard improved instrument adopted for use on official surveys has this equipment.

The essential features of this observation consist in the simultaneous determination of the true vertical and horizontal angles to the sun's center. The relation between the calculated azimuth of the sun and the recorded angle to the sun's center gives the true bearing of the fixed reference line. Any one of the following equations* may be entered, and the azimuth of the sun's center as referred to the true meridian at the epoch of observation, may be calculated:

$$\tan \frac{1}{2} A = \sqrt{\frac{\cos \frac{1}{2}(\zeta + \phi + \delta) \sin \frac{1}{2}(\zeta + \phi - \delta)}{\cos \frac{1}{2}(\zeta - \phi - \delta) \sin \frac{1}{2}(\zeta - \phi + \delta)}} \quad \ldots\ldots\ldots (1)$$

$$\cos \frac{1}{2} A = \sqrt{\frac{\sin S \sin (S - \text{co-declination})}{\sin \text{co-latitude} \sin \text{co-altitude}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which the "pole-zenith-sun" triangle is expressed as follows:

Pole to zenith $= 90° - \phi =$ co-latitude;
Pole to sun $= 90° - \delta =$ co-declination;
Zenith to sun $= 90° - h =$ co-altitude;
$S =$ one-half the sum of the three sides.

$$\cos A = \frac{\sin \delta}{\cos \phi \cos h} - \tan \phi \tan h \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

In case it is desired to obtain both time and azimuth, Equation (1) will be preferred. For a determination of azimuth only Equation (3) will be found to

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of the Public Lands of the United States", 1919, p. 104.

be the most convenient on account of the comparative ease of its reduction. An example of direct altitude observation for azimuth on the sun, north declination, using Equation (3) will be given. For a south declination, the function, "sin δ", would become negative by virtue of the sine of a negative angle being treated as negative in analytical reductions; should the algebraic sign of the result be positive, the azimuth, A, is referred to the north point; if negative, A is referred to the south point.

Table 1 gives a series of three direct altitude observations on the sun for azimuth. These observations were taken on August 2d, 1909, at the corner of Ts. 31 and 32 S., R. 43 and 44 W., 6th Prin. Mer., Colorado, in latitude 37° 15′ 05″ N.; longitude 102° 18′ 06″ N., at 7.30 A. M. apparent time. From a meridian determined with the solar attachment, an angle of 90° was turned to the east. On this line, a flag was set about 20 chains distant, from which as a reference point these observations were taken.

TABLE 1.—ALTITUDE OBSERVATIONS OF THE SUN FOR AZIMUTH.

Series of Three Observations.	Telescope.	Sun.	Watch Time.	Vertical Angle.	Horizontal Angle from Flag to Sun.
1st.	Direct	○+	7 h. 36 m. 54 s.	30° 05	0° 08′ 30″ to N
"	Reversed	+○	7 38 15	29 48	0 33 00 " "
	Mean			29° 56′ 30″	0° 20′ 45″ to N
2d.	Direct	○+	7 h. 41 m. 20 s.	30° 58′ 00″	0° 32′ 00″ to S
"	Reversed	+○	7 43 00	30 46 30	0 12 30 " "
	Mean			30° 52′ 15″	0° 22′ 15″ to S
3d	Direct	○+	7 h. 52 m. 00 s.	33° 05′ 00″	2° 11′ 00″ to S
"	Reversed	+○	7 53 48	32 53 30	1 50 00 " "
	Mean			32° 59′ 15″	2° 00′ 30″ to S

Reduce the mean observed vertical angle to the sun's center for each observation, to the true vertical angle, thus:

	1st Obsn.	2d Obsn.	3d Obsn.
$V =$	29° 56′ 30″	30° 52′ 15″	32° 59′ 15″
Refraction =	—1′ 40″	—1′ 36″	—1′ 28″
Parallax =	+ 8″	+ 8″	+ 8″
$h =$	29° 54′ 58″	30° 50′ 47″	32° 57′ 55″

Thence, reducing the first observation of the series for azimuth by Equation (3),

$$\cos A = \frac{\sin \delta}{\cos \phi \cos h} - \tan \phi \tan h.$$

The sun's declination for the mean period of three observations equals 17° 51′ 04″ N.

$$
\begin{array}{llll}
\log\cos\phi = 9.900674 & \log\sin\delta = 9.486493\ (+) & & \log\tan\phi = 9.881708 \\
\log\cos h = \underline{9.937897} & & & \log\tan h = \underline{9.759970} \\
\quad\ 9.838571 & \qquad\quad 9.838571 & \log & \quad = 9.641678 \\
 & \log \quad \overline{9.647922} & \text{nat}\ (-) & \quad = 0.43821 \\
 & \text{nat}\ (+)\ 0.44455 & & \\
 & \qquad (-)\ \underline{0.43821} & & \\
\end{array}
$$

$$\cos A = (+)\ 0.00634$$

A = true bearing of sun = N. 89° 38′ 12″ E.
Angle from sun to flag = (+) 0° 20′ 45″

True bearing of flag = N. 89° 58′ 57″ E.

A similar reduction may be made of the second and third observations, for the true bearing of the sun, from which :

By first observation, flag bears N. 89° 58′ 57″ E.
By second observation, flag bears N. 89° 58′ 26″ E.
By third observation, flag bears N. 89° 58′ 38″ E.

Mean true bearing of flag N. 89° 58′ 40″ E.

Hence, the indicated error of the solar attachment is 1′ 20″

At any place in the field however remote, the cadastral engineer of the present day is seldom without the means by which to determine true values of time, latitude, and azimuth which are necessary to a correct return of his survey, nor does he often fail to surround his methods with adequate verification in order to insure their required accuracy.

IDENTIFICATION AND RESTORATION OF CORNERS.

Permanency of Construction.

The boundary lines of the original survey must be retraced and the true bearings and distances between identified original monuments determined before the limits of the areas covered by existing entries and patents may be identified and the computation of the remaining subdivisions of public lands may proceed. Lost or obliterated corners are restored to their true original positions by a proper reference to the original field note record, influenced in some cases by competent collateral evidences which, before acceptance, however, must be justified in the light of the same record. Monuments restored in this manner, or those identified and reconstructed in place, no longer, as in former days, depend for permanency on shallow pits and mounds of earth with the mythical "charcoal deposit" or, in timbered and rocky regions, on small marked stones of indifferent composition and short-lived bearing trees.

When the true boundary is identified in a resurvey, after many retracements and much investigation, it is now possible to monument the corner permanently by virtue of the Act of Congress approved May 27th, 1908 (35 Stat., 347), and later Acts amplifying this authority. For this a concrete-filled, 3-ft., iron corner post is used. It may be 1, 2, or 3 in. in diameter, and is set in

the ground with the fill tamped in over a broad flange at the bottom. If the soil is sandy and loose, the flange of the post is set in a concrete base. This post is then witnessed by the best of available accessories, such as marked bearing trees, pits, or mounds of stone. The corner position is carefully re-monumented without destroying the physical evidence which served to identify its original position, the utmost regard being shown for all such evidences of original location. A complete record is kept of the description of the original marks as identified and all the resurvey additions thereto. The brass caps of the iron corner posts are, at the time used, suitably and plainly marked with steel dies, capital letters and Arabic figures being used. Fig. 5 shows a sample marking of brass caps. As an additional safeguard to insure the permanency of these monuments Congress, by an Act approved March 4th, 1909 (35 Stat., 1088, Sec. 57), has provided a fine and imprisonment for those who wilfully destroy them or effect their removal.

Recognition of Physical Evidence.

The Courts attach major importance to authentic evidence relating to the original position of an official corner monument, such evidence being given far greater weight than the technical record relating to bearings and lengths of lines. The legal signficance of original monuments makes it mandatory on the engineer to exercise the greatest diligence to find and restore each corner in its true original position. On the treeless prairies of the central Western States where original corners were generally constructed of wooden posts or small stakes with pits and mounds, the writer has identified and reconstructed hundreds of such monuments long obliterated and considered lost, by the ordinarily simple, but sometimes very expert, process of removing the top sod or soil with a sharpened spade and revealing the distinct outlines of the original pit beneath, in the exact form as left by the builder years before. The original pit has gradually filled with blowings and wash of a distinctly different nature from the homogeneous soil on all its sides. During many years of experience in this work, the writer has recovered indisputable outlines of such original pits under all conditions and kinds of soil—in roadways, on steep slopes, and on lands subject to overflow—and he has no hesitation in stating that when once dug if any part of the original sides of the pit still remains, the fill in the center can be identified with certainty by this method.

In timbered regions where Nature has healed the wounds made by the scribing tool of the original surveyor on the tree corner, or the "bearing" trees used as accessories to the corner, the expert will readily identify the new growth over the old blaze. A sharp axe applied above and below the growth will remove it in one piece, thereby uncovering the marks originally made on the tree itself and also those transferred to the piece removed, where they will appear raised in relief. Furthermore, removal of such growths from the old "line" trees and others of those which may mark the section line, will often reveal the old axe marks. This evidence when corroborated by a correct "ring count" in the growth itself will provide silent but convincing testimony

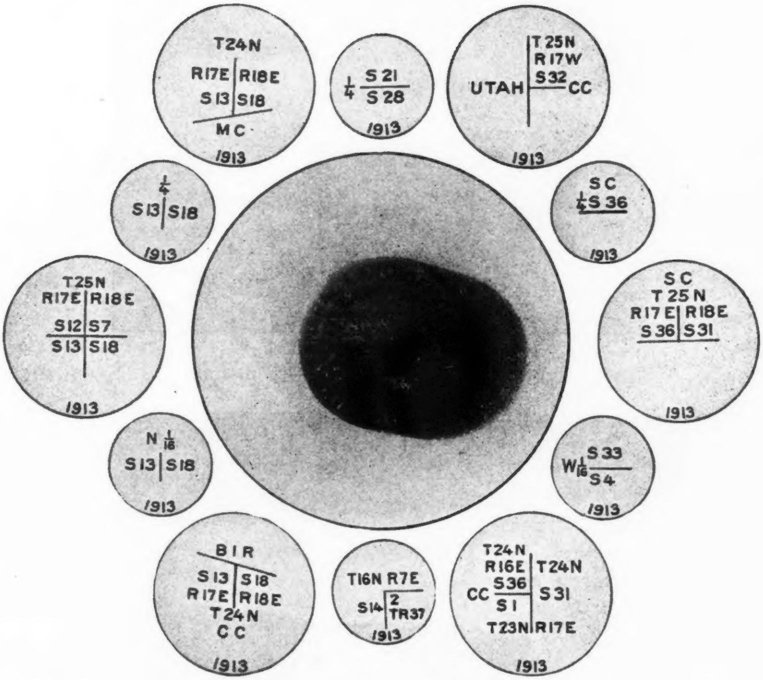


FIG. 5.—SAMPLE MARKINGS OF BRASS CAPS.

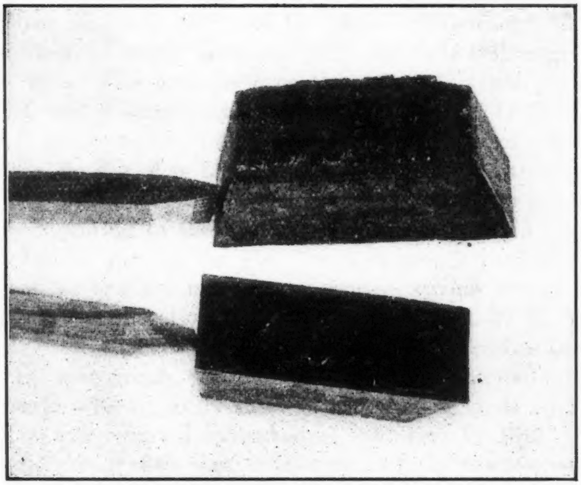


FIG. 6.—OVERGROWTH TAKEN FROM WHITE FIR TREE CORNER ON OREGON-CALIFORNIA STATE BOUNDARY.

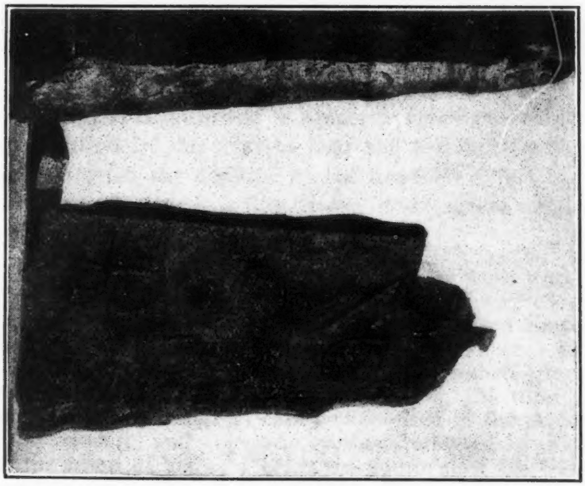


FIG. 7.—OVERGROWTH TAKEN FROM CYPRESS BEARING TREE IN FLORIDA.

as to the position of the original line even when the corner monuments themselves cannot be found. Illustrations of such growths, which were removed by the previously described method from the original blazes and on which the original marks appear in relief will be found in Figs. 6 and 7. Fig. 6 shows a chip from a white fir tree which in the original survey of the boundary line between the States of Oregon and California was marked and described by Deputy Surveyor Major in 1868, for the 147th-mile corner. This fir tree was readily identified 47 years later—in 1915—by cadastral engineers in a resurvey of this line. The growths over the original blazes were removed and the tree was found to be correctly marked as follows: 147 M on the east; O on the north; C on the south; and 42 L on the west faces of the tree.

The chip shown in Fig. 6 is from the west face of this tree. It plainly shows the original marks, "42L", in relief, and is made up of forty-seven growth rings corresponding to the interval in years which had elapsed since the date of the original survey. In Fig. 7 will be seen a part of the growth removed from over the original marks on a cypress-bearing tree at a meander corner on the line between Sections 32 and 33, T. 41 S., R. 37 E., Tallahassee Meridian, Florida. Originally, the thickness was a few inches greater than that shown in the photograph, the outside portion having been lost in the removal. The marks which plainly show in relief, were made on the tree in 1855, and the chip was removed by cadastral engineers in 1919. The "ring count" on the adjacent growth was "sixty-five", which was the correct time interval in years.

An interesting example of a restoration involving the identification of original bearing and line trees and the proper application of proportional measurement occurred in 1913 in the oil district of Louisiana in the determination of the section and subdivision of section lines of Section 3, T. 20 N., R. 16 W., L. M. The methods used were thoroughly tested in the trial of a boundary suit where a difference of 1.7 ft. involved a property value exceeding $800 000. Many days were required in the trial to hear the technical evidence alone. The following extract from the decision of the Supreme Court of Louisiana (No. 22 485, February 25th, 1918), is illustrative of the severe requirements of such restorations:

"Among the several surveys made, one stands out as most worthy of consideration by the Court. It was not made on behalf of any of the parties interested in this litigation, but it was ordered by the United States Government and it was executed under instructions from the General Land Office, whose stamp of approval has been placed upon it. Under these circumstances, the recognized ability and competency of the engineer, the total absence of any possible bias on his part, the great care he exercised in the performance of his work, the most modern and scientific methods adopted by him and the further fact that the result of his work bears the approval of the General Land Office, are, in our opinion, sufficient to establish a preponderance of evidence in favor of plaintiffs and to justify a decree based upon his findings under the law applicable in the case."

Methods of Restoration.

The original corners restored in a resurvey may be divided into two general classes: (*A*) those identified and reconstructed in place; and (*B*) those

restored by a mathematical or ethical procedure. Further defined, they will appear as follows:

(*A*).—A corner is identified and may be reconstructed in place:

1.—When the physical monument itself or any of its accessories are found and identified in their original positions.

2.—When the physical witness corner or any of its accessories are identified in position, from which the true point for corner is determined.

3.—When, although no physical trace of the monument or accessories remains, its original position has been perpetuated with certainty by settlers.

Referring to Paragraph 2, it naturally follows that, as the true point for the corner was not originally monumented, it is only necessary in such a case to identify the original witness monument in its original position and then proceed to the true corner point in accordance with the original record.

Identification under Paragraph 3 includes those instances where the original corner was at one time found and its true position perpetuated by some permanent mark of local origin, after which the physical evidences of the official monument were destroyed, and only for the permanent mark, the original position would have been lost.

(*B*).—A corner is identified and may be restored to place:

1.—When an appropriate reference is made to the original record, from which is established a definite relative position with respect to other adjacent and existing competent points of control, as follows: (*a*) other existing public land corners; or (*b*) corners of other systems of surveys or unchanged and definite original points of topography.

2.—When a position has been recognized for years as the location of the corner by all who reside in the vicinity, and has been agreed on and perpetuated as such by the several parties in interest; such point, however, must hold an acceptable relative position when compared with those original official monuments within the field of influence of which the point is found to be situated.

The method of Paragraph 2 under Heading, (*B*), does not imply necessarily that such position has any direct relation whatever to the true position of the corresponding original monument itself. Unusual cases will also arise where the nearest existing original corners are so far distant (all intervening monuments having been lost) as to lose in some measure their influence and value as factors of control. In this event, other competent collateral evidence will correspondingly increase in relative value for the identification of an acceptable position for the corner.

A choice must always be made between the several methods of restoration. The appropriate method will depend, of course, on local conditions existing in the particular case under consideration, selected so as to afford an equal protection for all the interests involved. To accomplish this result it does not always follow that the most probable position of the lost original monument

will be the one accepted when a proper consideration and combination of all available evidence is made. When the physical corner itself has once been destroyed and the original position lost, any method selected by which to restore it to its most probable position will not do so with certainty. It is better at once to adopt the position with respect to the nearest existing corners which has the greatest possible concordance with the original record, as all disposals of public lands are made in accordance with the original plat representing such record, both as to the position and extent of land intended to be conveyed. This is exemplified in the method ordinarily proper for the restoration of closing corners in cases where the closing line defines the boundaries of alienated lands. In such cases, the identification may be made by means of a proportional measurement based on the plat distances to the right and left of the closing corner, measured along the line closed on, rather than on the single-record connecting tie of the original surveyor alone; also, the same reasoning applies in the identification of those lost corners immediately preceding on the closing line, namely, the lost corners on a range line closing on a standard parallel, the identified positions of which ordinarily are based on a proportional measurement extending to the major line closed on (standard parallel), and not to the original closing monument itself, in cases where the monument is known and is off the major line.

Proportional Measurement.

The method given under Heading (*B*), Paragraph 1, is, in general, that method ordinarily referred to as "proportional measurement", by which is meant a measurement having the same ratio to that recorded in the original field notes as the length of the line by re-measurement bears to its length as given in the record. The ideal condition of course would be that in which the corners chosen as a basis for control would occupy in all directions the position they should with respect to each other without any excess or deficiency; however, such is rarely, if ever, the case. The evidences of position offered by these controls are always conflicting to a greater or less extent, but as they are of the same type and as controlling factors vary inversely as the distances involved, these evidences, although conflicting, may be consistently combined and, at the same time, admit of a practical and more or less general application. This has resulted in what is called the single and double proportionate method, on which it has been found, dependence must be placed in the execution of resurveys for the major number of restorations, and with respect to which the only general rule of procedure has been formulated.

An exception is made, however, to any such general rule of proportion in those cases where, on account of the existence of gross error and irrelation in the evidences of the original survey, or, in the presence of extensive obliteration, only one or only two of the nearest existing original corners have been used by entrymen for the control of locations where more controlling points should technically have been considered. The location of such a claim, so determined and perpetuated by improvements, will often differ materially from that determined by a proper consideration of all the existing corners, that is,

two or four. In such cases, it cannot generally be charged that a lack of good faith has been shown, and those acceptable positions for the lost corners as fixed by evidences of occupation are then adopted for the identification of the tract and possibly for other tracts which may be in harmony therewith. In these cases of irrelation natural conflicts will often be found to exist and must be recognized and identified on the ground by the engineer for adjudication by the proper Court. These locations are best identified in the form of tract segregations by metes and bounds in "independent" resurveys, further consideration of which will follow.

Theory of Non-Concurrent Parallel Forces.

On all lines of the same class in any survey, each call of the original record should bear, theoretically, its proportion of responsibility for discrepancies found to exist between evidences of such survey on the ground and the record thereof, and the errors should vary directly as the distance, along any line of such survey, of an identified control away from the locus of the lost corner. For example, if five or more monuments had been established originally, one of which is assumed to be lost, its restoration might be based on a consideration of as many temporary points as there are existing monuments, each temporary point to be assigned its proper value or weight, and to be located by proceeding on the record course and distance from one of the existing monuments. The temporary points thus determined might then be treated as a system of non-concurrent parallel forces acting normal to a horizontal plane, the position of the resultant of which is sought for the position of the lost corner. The restoration would be accomplished by assigning to each temporary point a value inversely proportional to its distance from the initial existing monument at the other end of the random line. Obviously, this academic reasoning could be applied in any practical restoration of public land corners only as between the nearest identified corners in any direction, in order to avoid a needless multiplicity of temporary points. If it could not be assumed that those corners which are situated outside of a certain restricted field of influence, are at so great a distance away as to have little or no effect, it at once becomes evident that a strict adherence to the foregoing principles would have no value for a practical application.

Fig. 8 illustrates the theory of non-concurrent parallel forces, as applied from a four-point control, for example:

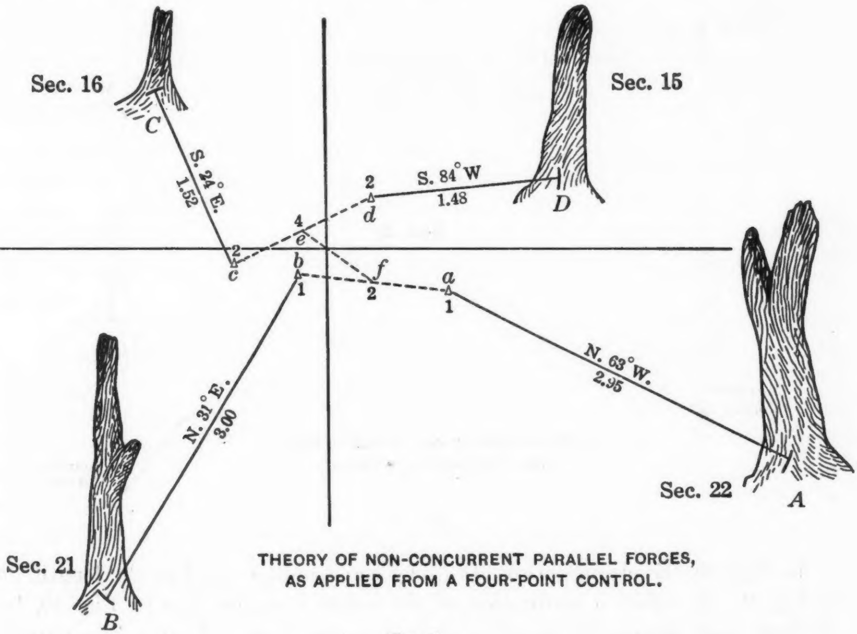
Let *A, B, C,* and *D* represent four identified bearing trees witnessing the position of a corner of the original survey; also, let *a, b, c,* and *d,* represent four corresponding temporary points for the restored corner, the positions of which are determined by the courses and lengths given in the original record as run from the identified original bearing trees, *A, B, C,* and *D*.

Since *C* and *D* are distant from *c* and *d* only one-half the distances of *A* and *B* from *a* and *b,* the temporary points *c* and *d* will be assigned a measure of influence (or weight) twice that of *a* and *b*.

Therefore, with *a* and *b* each valued as 1, *c* and *d* will each be rated as 2. As the values of *c* and *d* are equal, which is also true of *a* and *b,* the combined influence of each pair may be represented, as to *c* and *d* by

a temporary point, e, mid-distant on line cd, with a weight of 4, namely ($e = c + d = 2 + 2 = 4$), and as to a and b by a point, f, mid-distant on line ab, with a weight of 2, namely ($f = a + b = 1 + 1 = 2$). Thence, the true point for the restored corner which results, with due regard for the influence of each bearing tree, will lie on a line connecting e and f, a distance from f to e equal to two-thirds of the total, namely $\left(\frac{4}{2 + 4} = \frac{2}{3}\right)$.

The resultant point on the line, ef, will be removed therefore from e or f distances which assume an inverse ratio to the respective weights assigned thereto. Any combination of the points, a, b, c, and d, with properly assigned weights, will give the same resultant position for the true corner.



THEORY OF NON-CONCURRENT PARALLEL FORCES, AS APPLIED FROM A FOUR-POINT CONTROL.

FIG. 8.

Application of Theory to Broken Boundaries.—The restoration on the ground of any angle-point of a meander line, between the two nearest meander corners, would appear in the execution somewhat as shown in Fig. 9.

Let A and B represent, respectively, a re-established and a reconstructed original meander corner on the east and west boundaries of Section 28. The numbered lines, 1, 2, 3, and 4, and 6 and 5 represent consecutive meanders run out from B and A, respectively, on the record course and distance, to arrive at the temporary points, b and a. Then, the restored position for the desired angle-point will be on the line, ab, a distance from b represented by the fraction,

$$\frac{1 + 2 + 3 + 4}{1 + 2 + 3 + 4 + 5 + 6}$$

in which the numerals represent the record lengths only of the correspondingly numbered meander lines.

Similarly, all the angle-points of such a line may be reproduced on the ground by running out the complete record from either *B* or *A*, as shown graphically in Fig. 10.

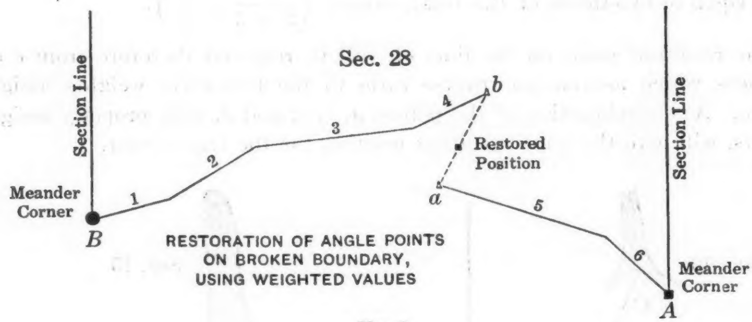


FIG. 9.

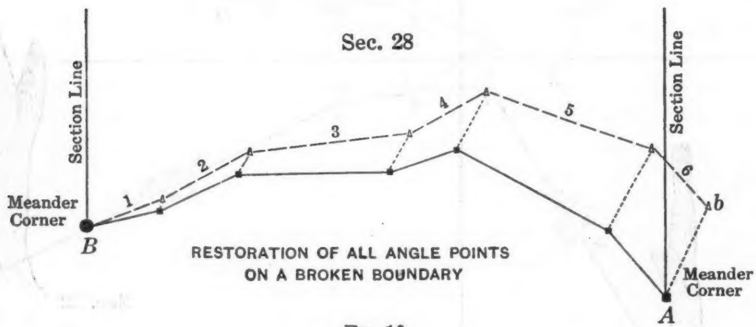


FIG. 10.

In Fig. 10, the closing error, *Ab*, is by construction equal to the length, *ab*, in Fig. 9. To effect a restoration of the entire meander line in Fig. 10, lay off from each temporary angle-point in the direction and on a line parallel to *Ab*, a distance which will be a proportional amount of the length of the closing error. This distance will be that one which is to the whole length of the error (*Ab*) as the distance of the particular temporary point from the initial point (*B*) is to the whole length of the meander line (from *B* to *b*). If one or both of the meander corners on the section lines are lost, they are first restored, before proceeding with the reproduction of the meander line.

Lost corners originally established in metes and bounds surveys, and all other irregular surveys not of the regular rectangular system, broken controlling boundaries, and the angle-points of a meander line, are restored and reproduced on the ground according to the principles as described. A valuable discussion* by Leonard S. Smith, M. Am. Soc. C. E., of a similar method of restoration of a broken boundary in which the problem was the retracement

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 429.

of an old road, will be found to be of great interest to those in contact with this subject. In this case, the "twist" or error in azimuth of the original surveyor's meridian was computed and also the ratio of the lengths of the original and the resurvey chains.

Oklahoma-Texas Boundary Controversy.—A very important boundary dispute of related interest, now in litigation, and one in which the U. S. Cadastral Engineering Service is making extensive cadastral and topographic surveys for the preparation of the necessary maps, is that which involves the determination of the true location of the boundary line between the States of Texas and Oklahoma along the Red River. The discovery of oil within the river flood plane has precipitated a complex dispute as to the ownership of the river bed. The State of Oklahoma in the October Term, 1919, filed a suit in equity, Original No. 27, in the Supreme Court of the United States *versus* the State of Texas, Defendant. The question at issue involves the construction of the Treaty of 1819, between the United States and Spain, as to whether the specifications of the treaty refer to the south bank of the Red River or the middle thread of the main channel—the treaty, of course, referring only to the river in the position it occupied in 1819. In the determination of this position, the comparatively new science of Ecology* will have an important bearing, as well as geological and topographical considerations. Riparian rights such as are appropriate in case of accretion or avulsion might then be applied.

In March, 1920, the U. S. Department of Justice filed a motion to intervene in the suit, in order to conserve the rights of the United States to possible public domain and to protect the rights of the Indian wards of the Government within the river basin. The Supreme Court, in April, 1920, granted this motion, enjoined the State of Texas from selling any purported rights covering any lands or parts of the river bed lying north of the line of the south bank as it existed in 1819, and appointed a receiver to take over the operation of the oil field in certain ranges and between the south bank and middle thread of the main channel of the river. A suit was subsequently filed in the U. S. Federal Court for the Western District of Oklahoma, by the Department of Justice, in behalf of the Indian allottees, in order to protect riparian claims attached to Indian allotments on the north bank and extending to the medial line between the river banks (the present meander lines of the right and left banks), which according to the Indian treaty of 1867, is the line designated as the south boundary of the Kiowa and Comanche Reservations. The District Court has appointed a receiver to take over the operation of the oil field within the riparian claims of the Indians. The receiver for the Supreme Court under a working agreement has control of any areas in conflict between the two receiverships.† Separate consideration will be given in these suits to the questions of fact as to the positions determined for the true boundaries at the definite dates. A reference to Figs. 11 and 12 will give the reader a very good idea of the appearance

* Plant Oecology: the Adaptation of Plants to Life Under Particular Environing Conditions; one of the chief exponents of the science in the United States is Dr. Henry C. Cowles of the University of Chicago.

† Report of the Commr. of the General Land Office to the Secretary of the Interior, June 30th, 1920, pp. 33–34.

of the oil field under the receivership. Fig. 11 is a view of the Red River, south of T. 5 S., R. 14 W., I. M., Oklahoma, looking north from the Texas side. The irregular full line indicates the possible position of the 1819 river bank, and the dashed line shows the southern limit of the area under the Supreme Court Receivership. Fig. 12 is a view of the Red River, south of T. 5 S., R. 14 W., I. M., Oklahoma, looking northeast from the Texas side. The full line indicates the possible position of the 1819 river bank. The dashed line shows the southern limit of the area under the Supreme Court Receivership. The bed and flood-plain of the river are located over an oil "dome" or pool, and nearly every well has been a producer and property values exceeding $100 000 000 are involved in this boundary litigation. The photographs were taken during 1920.

Erroneous and Fraudulent Meander Lines.—Meander lines as shown on approved plats are sometimes erroneous or fraudulent. In these cases the lines as recorded do not and never did conform even approximately to the mean high-water elevation of the actual body of water, and, thus becoming fixed boundaries, they sometimes omit large areas of public land from the original surveys. The location of a true mean high-water mark as of an early date, for critical comparison with the restored record position of the original meander line, overlaps somewhat into the realms of geology and ecology. These questions while involving the tracing of old escarpments in the soil and investigations into the age and growth of trees, are not overlooked by the cadastral engineer.

In the practical application of the foregoing mathematical method of restoration, which contemplates the use of both lengths and courses of original lines and utilizes weighted values, to the cardinal lines of the rectangular system of original surveys, the necessary introduction of three or four temporary points, on which to base a final determination of a restored position, complicates the computations and the record of the field operations to such an extent that the small advantage which the method possesses as to exactitude ordinarily does not justify its use in preference to the more adaptable and less complicated single and double proportionate methods of restoration. Except in cases where excessive distortion is developed, all restorations along the lines of the rectangular system will yield readily and logically to the principles involved in the latter methods. Although both the "weighted value" and the "double proportionate" methods depend on the same general underlying principles, the latter, although less theoretical, will produce results equally as good and is more practical of application than the former method, and has been adopted for official use.

Single and Double Proportionate Methods.

The single and double proportionate methods of restoration are based on a consideration of the record distances only, the record courses in all cases being ignored, the measured distance between the two nearest existing corners being divided into distances proportional to those given in the record between the same corners. A necessary advantage of this method over that of the "weighted values" is that the restoration of one lost corner may be based on

FIG. 11.—RED RIVER, LOOKING NORTH FROM TEXAS SIDE. IRREGULAR FULL LINE INDICATES POSSIBLE POSITION 1819 RIVER BANK. DASHED LINE SHOWS SOUTHERN LIMIT OF AREA UNDER SUPREME COURT RECEIVERSHIP.

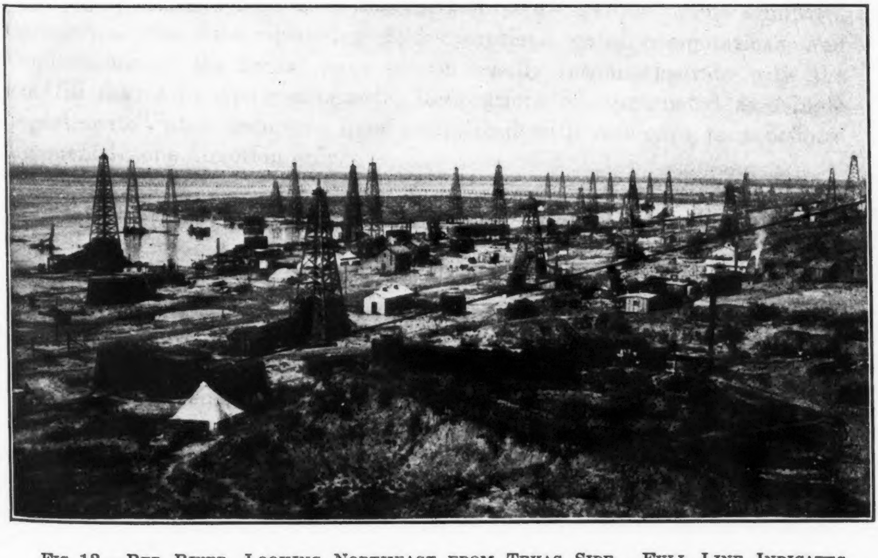


FIG. 12.—RED RIVER, LOOKING NORTHEAST FROM TEXAS SIDE. FULL LINE INDICATES POSSIBLE POSITION 1819 RIVER BANK. DASHED LINE SHOWS SOUTHERN LIMIT AREA UNDER SUPREME COURT RECEIVERSHIP.

the restored position of any other lost corner without producing the incongruities which will develop occasionally in using the method by "weighted values".

In explanation of the "double proportionate" method as now adopted for official use, assume that the problem is the restoration, from a four-point original control, of a lost section corner on the rectangular subdivisional lines of a township survey. A retracement is first made on the meridional line connecting two of the original corners, on which line a temporary stake is set at the proper proportionate distance, based on the original record. This point will determine the latitude of the lost corner. Next, the two original corners remaining are connected by a latitudinal retracement and a corresponding temporary stake set thereon, determined by proportionate measurement in a similar manner. The second point determines the position of the lost corner in departure. The intersection of two cardinal offset lines run from these two temporary stakes, north, south, east, or west, as relative situations may determine, will then define the restored position of the lost corner. The obliquity of the section lines in their restored positions, such as occasionally results, suggested some years ago the application of a slightly different method of proportionment based on those field distances which are measured along the hypotenuses of the several right-angled triangles created instead of along the long legs. This method when utilized provides an interesting and intricate mathematical study and, in the past, has required of the writer many days of harrowing field computations for its application. However, the impracticability of this method for general use is now admitted, inasmuch as the time-consuming field operations, extra computations, and complications of the record have proven wholly incommensurate with the doubtful degree in equity attained. Restorations are designated as "single proportionate" when made on lines established with reference to a definite alignment in one direction only.

IDENTIFICATION OF ALIENATED LANDS.

General Principles.

The fundamental principles used for the protection of *bona-fide* rights under the law are identical in all types of resurveys. In the case of the identification of the original position of any tract of land to which private rights have attached under the original survey, the identification when completed is regarded as a demonstration on the part of the General Land Office, in the light of the best evidence available, of the original position of those entered or patented subdivisions which are included in the description of the entry or patent, all as referred to the original survey. The cadastral resurvey, as far as private rights are concerned, has no creative function of its own, and it cannot change or supersede a completed function of any monument of the original survey. It must accept, without change, these original evidences of location as they are found to exist. Before a settler who "squats" or establishes residence on unsurveyed land, may make entry in accordance with

the legal subdivisions of an official plat after the land has been surveyed, he is required to adjust his lands to the descriptions and lines of the official survey. The settler who establishes himself on lands already subdivided and platted has the advantage of the "squatter", in that he has the original corners and an original record at his disposal to facilitate the proper location of his lands. Naturally, no less is required of the entryman on surveyed lands, who has monuments and an original record to guide him, than is required of the "squatter" on unsurveyed lands who has none; for the lands of the former are also required to bear an acceptable harmonious relation to authentic evidences of the original survey, in accordance with which his entry was made. For example, if a settler makes entry for lands described in Section 5 and then establishes his residence and erects improvements not on Section 5, but somewhere on Section 7 or Section 17, his location is clearly erroneous under his original description, and, therefore, the description of his lands must be changed if he is to obtain a clear title to those legal subdivisions actually occupied. If a patent has been issued under the erroneous description, a corrected patent may be issued on a proper showing in lieu of the original. This may only be accomplished in accordance with the regulations issued under authority of the legislation providing for amendment of entries (Act of February 24th, 1909, 35 Stat., 645).

Bona-Fide Rights

Thus, the cadastral engineer of the Government draws a clear distinction between the *bona-fide* rights as to location and as to occupation. Only with the former is he directly concerned, for the rights as to occupation are without his province and are entirely within the jurisdiction of the adjudicators of the proper court of law. If an entryman finds that his lands are erroneously located, his relief may be found only in accordance with existing law and not in any particular inspiration of a surveyor. However, in the case of irrelation and obliteration of corners of the original survey, where any one of several possible original positions is acceptable, that acceptable position is used which more nearly than any other covers the lands actually occupied and improved; in other words, the improvements are taken to indicate the particular original corner or corners which are to be used to control the position of the lands. The definition of good faith in location, as furnished the cadastral engineer by the Commissioner of the General Land Office, is necessarily broad, as witness the following:*

"It may be held generally that an entryman has located his lands in good faith * * * when it is evident that his interpretation of the record of the original survey as related to the nearest existing corners at the time the lands were located (as defined by his fencing, culture, or other improvements) is indicative of such a degree of care and diligence upon his part, or that of his surveyor, in the ascertainment of his boundaries, as might be expected in the exercise of ordinary intelligence under existing conditions."

* "Advance Sheets of a Revision of the Manual of Instructions for the Survey of the Public Lands of the United States," Commissioner of General Land Office, June 16th, 1919, p. 278.

In the application of this definition to field conditions, the U. S. Cadastral Engineer has become properly impressed with the purpose of his task and the stability and dignity which is attached to a work so great and important, commensurate with the broad foundation in science and law. The records of a cadastral resurvey must form an enduring basis on which depends the security of the title to all lands acquired thereunder, and the field notes must be prepared so that under the test of the closest possible scrutiny at all times, present and future, the record can be regarded as conclusive in the matter of the proper location of private rights.

General Types of Resurvey

In general, any field condition that may arise will yield properly to the application of either of two recognized methods of executing Government resurveys, namely, the dependent or independent resurvey.

The Dependent Resurvey.—The typical dependent resurvey may be defined as an official re-marking on the ground of all the original section lines in their original positions, in accordance with the best available evidences thereof, in such a manner that all original subdivisional units, as identified, will occupy those positions which result from a proper legal subdivision of the sections thus restored. It is obvious that this type of resurvey is chiefly applicable to those cases showing fairly a concordant relation between conditions on the ground and the record of the original survey, for titles, legal areas, and descriptions are maintained as unchanged. The primary control for such a resurvey is based, first, on identified existing corners of the original survey and other acceptable points of control; and, second, on the restoration of missing corners by proportionate measurement in harmony with the original record.

The Independent Resurvey.—On the other hand in areas where gross error or irrelation is present, as between the existing evidences of the original survey, a zone of uncertainty often exists wherein the strict application of proportional measurement is either impossible or entirely inadequate for the protection of rights acquired in good faith under the law. In such cases, segregation of private land claims by metes and bounds surveys with a suitable tract number, from a one-point control according to the original record, is often necessary, or the conformation of such claims to the lines of an independent resurvey under either the original or new legal subdivisions, as may be appropriate; in either case, however, the claim as identified will occupy an acceptable position when referred to suitable evidences of the original survey. In such cases, there is generally no necessity whatever of applying any original restorative process for the description of the remaining unclaimed public domain, and an independent resurvey is executed in accordance with the cardinal lines of the official rectangular method of subdivision which, as to the vacant public domain, then supersedes the record of the original survey. In this type of resurvey, in cases where original descriptions are changed, the descriptions under the resurvey (tract segregations, or legal sectional subdivisions) and under the original survey of all private claims are appropriately shown, cross-referenced in an index table on the plat, and, in all

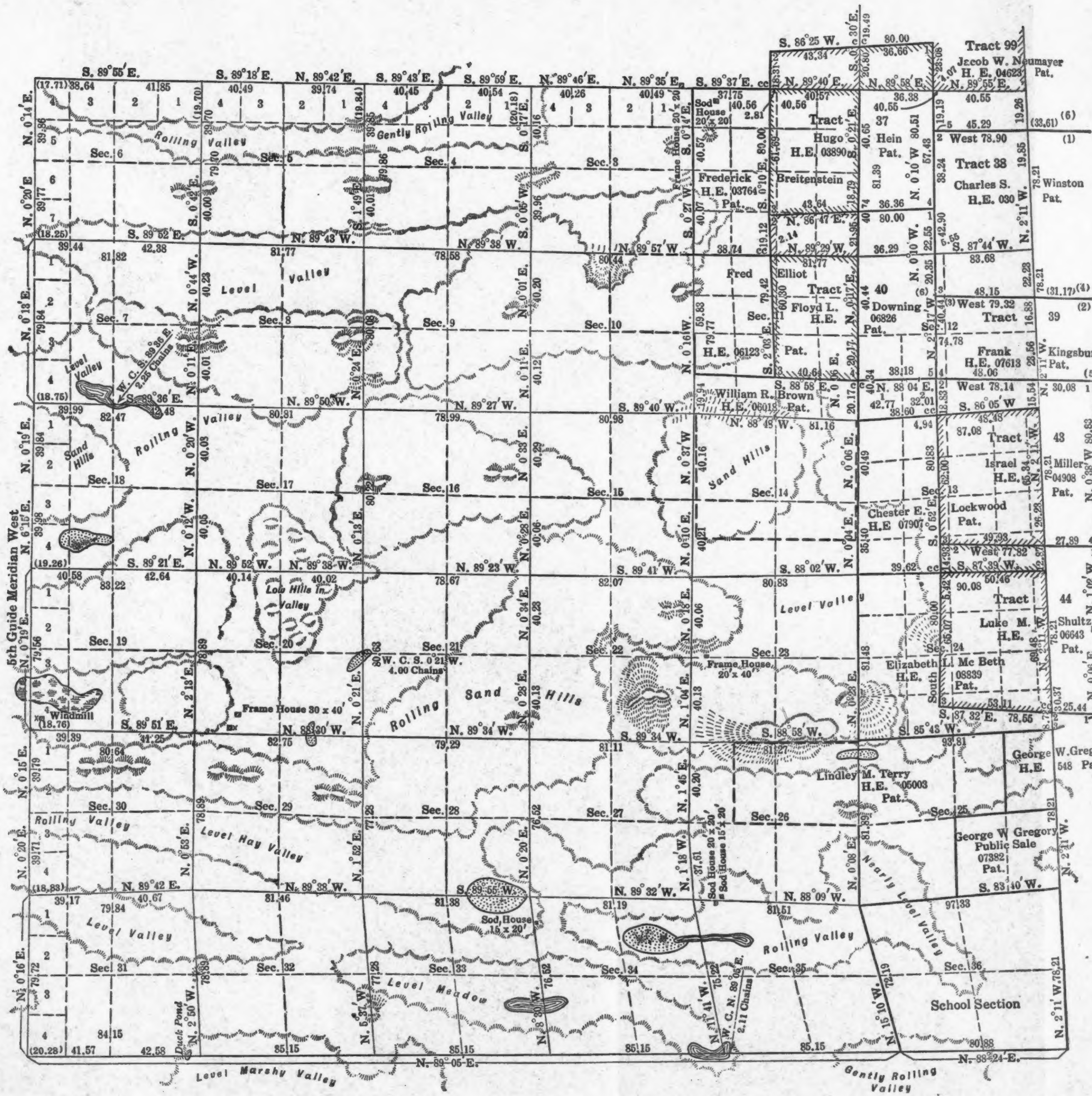
cases, existing private claims are blocked out in place and designated on the plat for further identification.

With respect to the protection of *bona-fide* rights, the basic principle is identical in either the dependent or independent type. In the former, all the lands patented and entered are identified theoretically by the legal subdivisions of the exhibited sections; in the latter, the identification is accomplished either by tract segregations or by conformations of the private claims to the sectional subdivisions, under either the old or new descriptions.

On Plate VI is shown the resurvey of T. 27 N., R. 40 W., 6th P. M., Nebraska, which embraces boundary lines exemplifying both the dependent and independent types of resurvey, the limiting boundary between the two being along the original eastern boundaries of Sections 3, 10, 14, 23, 26, and 35, and the northern boundary of Section 14. The sectional lines in the dependent portion are covered by a resurvey, as shown on the plat approved October 4th, 1883, in their original positions according to the best available evidence of the positions of the original corners; all differences between the measurements shown on the original plat and those derived in the retracements have been distributed proportionately between accepted corners in accordance with surveying rules. The sectional lines shown in the independent part are not established on cardinal courses with regular corner spacing as is usually the case when this method is utilized, for by reason of a condition peculiar to this locality they were retained as delineated in the field, on a basis of proportional measurement against the "limiting" boundary. The plat does furnish, however, a normal example of the conformation of several entries to the lines of the resurvey under their original descriptions, and also exhibits various metes and bounds surveys which have been executed by the use of a one or two-point control. In Table 2 will be found the necessary information as to the segregated tracts, and in Table 3 areas of the various conflicts and exclusions are given. The method shown of treating the areas in conflict provides a comprehensive basis for subsequent adjudication of the private rights involved, whether to all or any part of the lands in controversy. In the particular locality under discussion, settlement of the litigation resulting from the strife of the several boundary disputes was suspended by the local Court until the plats of the resurvey became available.

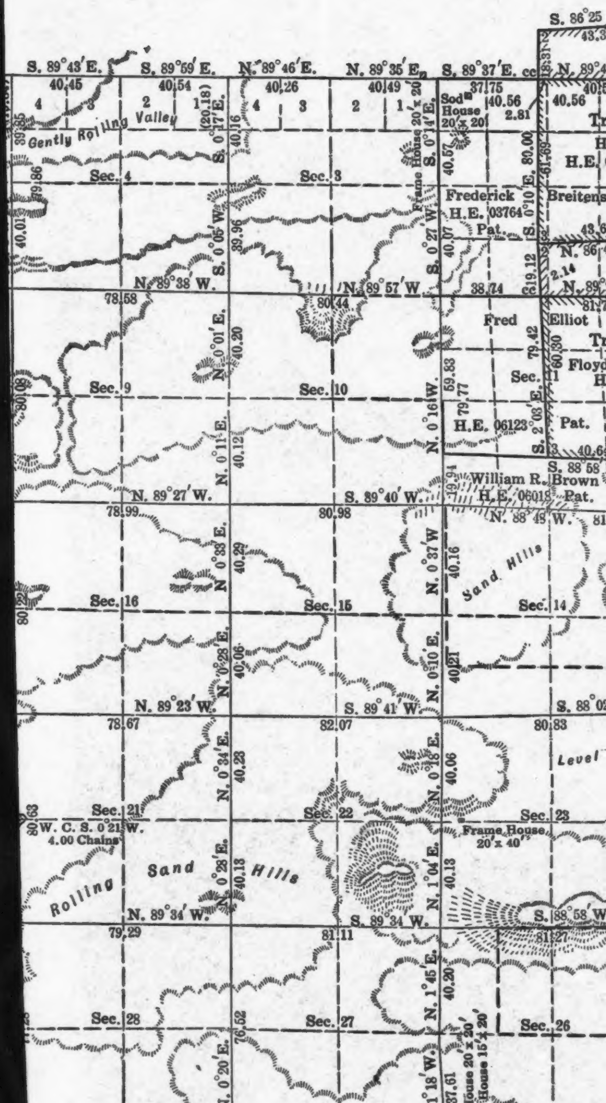
TABLE 2.—PLAT OF RESURVEY IN NEBRASKA, SEGREGATED TRACTS.

Resurvey.	Kind.	Entry.	Entryman.	Description under Original Survey.	Sec.	T. N.	R. W.	Status.
Tract 99...	H. E.	04623	Jacob W. Neumayer	S½. Sec. 31, T 28 N., R. 39 W., and N½	6	27	39	Patented
Tract 37...	H. E.	03890	Hugo Hein	All of	1	27	40	Patented
Tract 38...	H. E.	030	Charles S. Winston	S½, Sec. 6, and N½,	7	27	39	Patented
Tract 39...	H. E.	07613	Frank Kingsbury	S½, Sec. 7, and SW¼,	8	27	39	Patented
Tract 40...	H. E.	06826	Floyd L. Downing	All of	12	27	40	Patented
Tract 43...	H. E.	04908	Israel Miller	All of	18	27	39	Patented
Tract 44...	H. E.	06643	Luke M. Shultz	All of	19	27	39	Patented



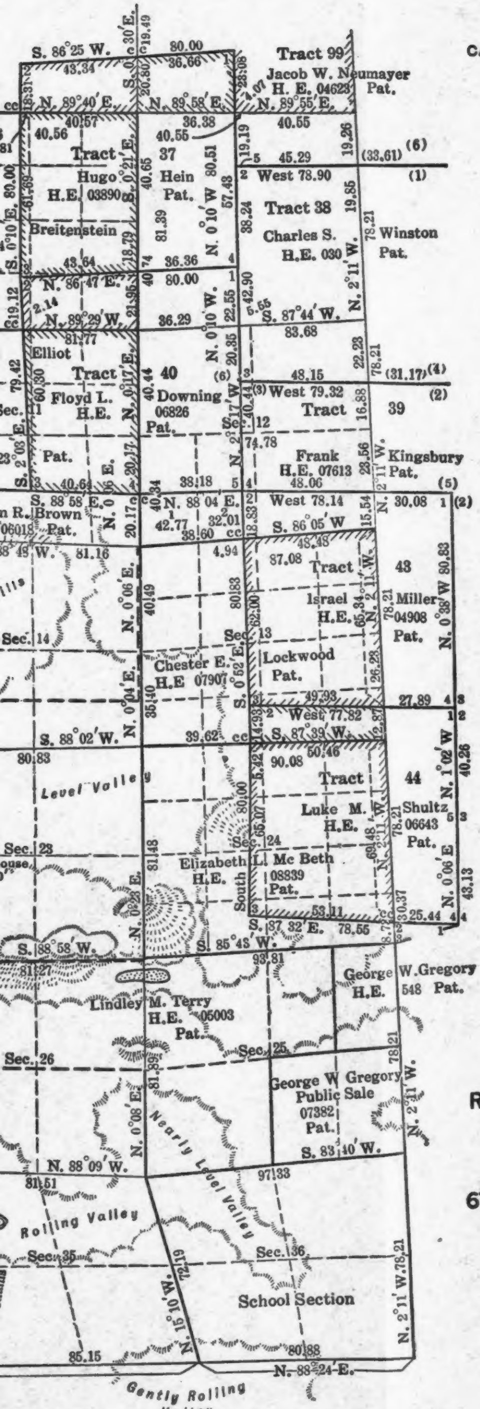
RESURVEY OF TOWNSHIP NO. 27 NORTH RANGE NO. 40 WEST 6TH PRINCIPAL MERIDIAN, NEBRASKA

Scale in Chains
0 10 20 40 80



S. 89°43′E.
S. 89°59′E.
N. 89°46′E.
N. 89°35′E.
S. 89°37′E.
S. 86°25
Gently Rolling Valley
Sec. 4
Sec. 3
Frame House 20′x 20
Sod House 20′x 20′
Frederick
H.E. 03764
Pat.
Breitens
N. 89°38′W.
N. 89°57′W.
Fred
Elliot
Floyd
Sec. 9
Sec. 10
Sec. 11
H.E. 06123
Pat.
William R. Brown
H.E. 06018 Pat.
N. 89°27′W.
S. 89°40′W.
N. 88°48′W.
Sand Hills
Sec. 16
Sec. 15
Sec. 14
N. 89°23′W.
S. 89°41′W.
S. 88°02
Level
Sec. 21
Sec. 22
Sec. 23
W. C. S. 0°21′W.
4.00 Chains
Frame House 20′x 40′
Rolling Sand Hills
N. 89°34′W.
S. 89°34′W.
S. 88°58′W.
Sec. 28
Sec. 27
Sec. 26
Sod House 20′x 20′
Sod House 15′x 20′
S. 89°55′W.
N. 89°32′W.
Sod House 15′x 20′
Rolling
Sec. 33
Sec. 34
Sec. 35
Level Meadow
W. C. N. 89°05′E.
2.11 Chains
N. 89°05′E.

PLATE VI.
PAPERS, AM. SOC. C. E.
NOVEMBER, 1921.
FARNSWORTH ON
CADASTRAL RESURVEYS.



RESURVEY OF TOWNSHIP
NO. 27 NORTH
RANGE NO. 40 WEST
6TH PRINCIPAL MERIDIAN,
NEBRASKA

Scale in Chains
0 10 20 40 80

TABLE 3.—PLAT OF RESURVEY IN NEBRASKA, AREAS OF CONFLICTS AND EXCLUSIONS.

	In Conflict with:	Area, in acres.		Exclusive of Conflict with:	Area, in acres.
Tract 37...	Sec. 36, T. 28 N., R. 40 W.	80.18	Tract 37	See. 36, T. 28 N., R. 40 W.	560.68
Tract 37...	Indem. Sel. List 2, S½, Sec. 35, T. 28 N., R. 40 W.	84.80	Tract 37	Indem. Sel. List 2, S½, Sec. 35, T. 28 N., R. 40 W.	556.06
Tract 37...	Sec. 2, T. 27 N., R 40 W.	263.22	Tract 37	Sec 2, T. 27 N., R. 40 W.	377.64
Tract 37...	All lands shown above	428.20	Tract 37	All land shown above	212.66
Sec. 2.....	Tract 37	263.22	Sec. 2	Tract 37	394.60
Sec. 2.....	Tract 40	88.90	Sec. 2	Tract 40	568.92
Sec. 2.....	Tracts 37 and 40	352.12	Sec. 2	Tracts 37 and 40	305.70
Tract 40...	Sec. 2	88.90	Tract 40	Sec. 2	557.70
Tract 40...	H.E.06123 as to Sec. 11	252.60	Tract 40	H. E. 06123 as to Sec. 11	394.00
Tract 40...	Sec. 2 and H.E. 06123	341.50	Tract 40	Sec. 2 and H. E. 06123	305.10
H.E.06123.	Tract 40	252.60	H.E.06123	Tract 40 (as to Sec. 11 only)	238.38
Tract 43...	Sec. 13. T. 27 N., R. 40 W.	313.12	Tract 43	Sec. 13	317.18
Sec. 13....	Tract 43	313.12	Sec. 13	Tract 43	368.88
Sec. 13....	Tract 44	69.72	Sec. 13	Tract 44	612.28
Sec. 13....	Tracts 43 and 44	382.84	Sec. 13	Tracts 43 and 44	299.16
Tract 44...	Sec. 13	69.72	Tract 44	Sec. 13	570.36
Tract 44...	Sec. 24	347.91	Tract 44	Sec. 24	292.17
Tract 44...	Secs. 13 and 24	417.63	Tract 44	Secs. 13 and 24	222.45
Sec. 24....	Tract 44	347.91	Sec. 24	Tract 44	385.29

JURISDICTION.

The extent of the jurisdiction and authority of the Cadastral Engineering Service through the Commissioner of the General Land Office should be clearly understood. Section 453 of the Revised Statutes, reads, in part, as follows:

"The Commissioner of the General Land Office shall perform, under the direction of the Secretary of the Interior, all executive duties appertaining to the surveying and sale of the public lands of the United States, or in any wise respecting such public lands; and, also, such as relate to private claims of lands, and the issuing of patents for all grants of lands under the authority of the Government."

The Act approved March 3d, 1899 (30 Stat., 1097), re-affirms:

"That hereafter all standard, meander, township, and section lines of the public land surveys shall, as heretofore, be established under the direction and supervision of the Commissioner of the General Land Office, whether the lands to be surveyed are within or without reservations, * * *."

The Acts of March 3d, 1909 (35 Stat., 845), and of September 21st, 1918 (40 Stat., 965), provide in part:

"That no such resurvey or retracement shall be so executed as to impair the *bona-fide* rights or claims of any claimant, entryman, or owner of lands affected by such resurvey or retracement."

From this it is clear that, in the exercise of his authority, the Commissioner of the General Land Office shall cause all resurveys to be executed in such a manner as to afford a fair and equitable identification of the original boundaries of all private claims, so that rights acquired in good faith shall be unimpaired. If title has not finally passed from the Government, the Commissioner is clothed with the authority to perform any and all acts incident, appertaining to, or necessary for, the disposal of the lands. Where a patent has been issued,

the jurisdiction of the Land Department over the lands patented terminates, and its power to examine and decide on claims to such lands becomes exhausted. *Bona-fide* rights, however, can be affected by a resurvey only in the matter of position or location on the earth's surface, and the cadastral engineer is concerned only with the question as to whether lands covered by such rights have actually been located in good faith. Other questions of good faith, such as priority of occupation, possession, continuous residence, value of improvements, and cultivation, when considered apart from the question of the position of the original survey, do not affect in any manner the problem of resurvey.

The Commissioner of the General Land Office is well qualified, and is authorized, to identify and indicate the original positions of all lands, as described in outstanding patents, and to delineate the boundaries of the public lands remaining, irrespective of whether the lands thus identified are actually occupied by a patentee or his successor in interest. However, in case of either patented or unpatented lands, where an owner or entryman seeks to maintain a claim that his rights as to location have been impaired by a resurvey, he will submit a proper protest, and the case will still remain within the jurisdiction of the General Land Office (subject, however, to the right of appeal to the Secretary of the Interior), which office after a proper consideration of the protest will reach a final adjudication of the dispute.

It will thus be realized that, by virtue of the legislation now existing, in any and all areas where uncertainties of the true locations of lands obtain, especially in those regions ridden with boundary disputes, strife, and litigation, the application of the modern resurvey procedure and the subsequent filing of a competent plat, will provide a necessary and enduring basis, not only for a comprehensive adjudication of the private rights involved, but also for the disposal of any public lands which may be found therein.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE FLOOD OF SEPTEMBER, 1921, AT SAN ANTONIO, TEXAS.*

By C. Terrell Bartlett,† M. Am. Soc. C. E.

Synopsis.

This paper describes the flood of September 9th, 1921, in the San Antonio River, at San Antonio, Tex., and in several tributary creeks all of which traverse that city, the fundamental conditions of topography and climate, and the meteorological factors and hydraulic elements of the flood in so far as it was possible to determine them.

Before describing in detail the recent flood experienced by the City of San Antonio, and its engineering aspects, it will be well to outline briefly the principal geographic, topographic, and climatic features of the region.

Geographically and geologically, the State of Texas is divided into three grand provinces as shown in Fig. 1. First, is the Rio Grande Plain, or Coastal Plain, an alluvial prairie extending from the boundaries of Louisiana and Eastern Oklahoma to the Rio Grande, and from the Gulf of Mexico inland from 150 to 200 miles. The topography ranges from the coastal flats through gently undulating and rolling country to altitudes of 400, 600, and 1 000 ft. along the inner margin of this region as one follows it from the Red River south and west to the Rio Grande.

The second of these great provinces extends from the margins of the Coastal Plain west to the Pecos River, the New Mexico line, and, northward, it embraces the Panhandle. This is the Central Plain, and forms the southern extension of the great plains of the Dakotas, Kansas, and Eastern Colorado. The third division known as Trans-Pecos Texas is part of the Rocky Mountain Region.

The line of division between the Rio Grande Plain and the upland plain or plateau is marked sharply and definitely by a great geologic fault which runs south about 300 miles from the Red River on the northern boundary of the State and thence swings west for 150 miles where it crosses the Rio Grande into Old Mexico. This fault is known as the "Balcones Escarpment", and is characterized by a sudden rise in altitude from the Coastal Plain to the

* Presented at the meeting of October 5th, 1921.
† San Antonio, Tex.

plateau region. At Del Rio, where this escarpment crosses the Rio Grande, the upper plain is about 1 200 ft. higher than the Rio Grande Plain, and the rise is very abrupt. In the vicinity of Fort Worth, the elevation is not more than 300 ft. and rises in a gradual incline across 15 to 20 miles.

The mean annual rainfall of Eastern Texas along the Louisiana line is 50 in. Thence, westward, it decreases to 30 in. along the eastern margin of the plateau, or Central Plain, and westward and southward to 20 in. at Del Rio and Brownsville. Although the influence of altitude is not apparent in the mean annual rainfall of Texas, it has long been recognized that in many cases the sudden rise at the Balcones Escarpment causes intense precipitation along and just above its margin. Due to the proximity of the Gulf, intense rains are also frequent along the immediate coast.

San Antonio lies in the Rio Grande Plain immediately below the Balcones

FIG. 1.

fault zone, adjacent to the great angle where the plateau juts out into the plain. From the northern limits of the city, the country rises to the northwest for about 12 miles to the elevation of the plateau, 700 ft. above the general level of the city. On this sloping margin of the plateau are three comparatively small water-sheds the streams of which traverse the city and unite near its southern limits as shown in Fig. 2. All the streams were in severe flood on the night of September 9th, 1921.

The City of San Antonio occupies 36 sq. miles and comprises a central valley the elevation of which is 650 ft. above sea level. It is bordered on the east and north by ridges about 100 to 150 ft. above the center of the city, and on the southwest by a slightly elevated plain. By the census of 1920, the population was 162 000.

The San Antonio River rises from fissure springs near the northern city line, about a mile east of its center, and winds through the heart of the city

within a well-defined flood-plain from ½ to ¾ mile in width. The flow from these springs ordinarily fluctuates between 50 and 150 sec-ft. The length of valley within the city limits is 6.3 miles, and the length of the river channel is 11.9 miles, with a fall across the city of approximately 90 ft. The greater

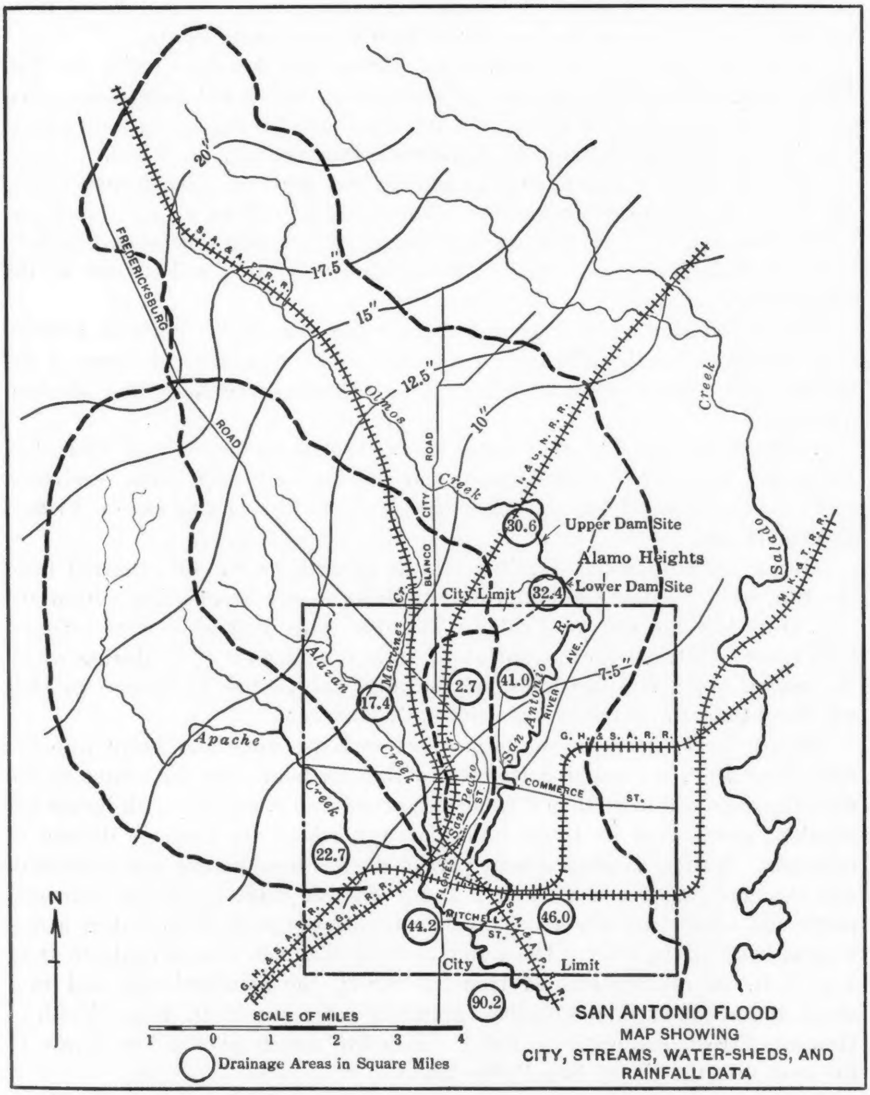


FIG. 2.

part of the business district, some of the industries, and the older and more thickly settled residence districts, lie in the immediate river valley.

The largest of these three water-sheds tributary to the San Antonio River lies north of the city, and is drained by a water-course known as Olmos

Creek which is ordinarily dry. The Olmos water-shed is 32.4 sq. miles in extent which, with the lower area tributary to the river proper, makes a total area of 41 sq. miles above the center of the city.

Within the city and 2 miles north of its center, San Pedro Creek rises from springs the flow of which is seldom greater than 10 sec-ft. Above these springs is a small water-shed entirely within the corporate limits.

Above the successive confluences of Alazan and Apache Creeks, the San Pedro Creek has a tributary area of only 2.7 sq. miles, but when augmented by these water-courses, it has a total drainage area of 44.2 sq. miles where it empties into the San Antonio River near the southern city limit line.

Both the Alazan and Apache Creeks are dry arroyos. The former enters the city from the northwest and has a water-shed of 17.4 sq. miles. The latter enters from the west and has a water-shed of 22.7 sq. miles, measured, in both cases, to their junctions with San Pedro Creek about 1½ miles south of the center of the city.

The Lower San Pedro traverses the western part of the business district, and, together with the Alazan and Apache Creeks, runs through some of the cheaper and more thickly settled residence districts, especially the Mexican quarter.

A drouth of several months' duration was broken on the night of Thursday, September 8th, 1921. San Antonio and the region to the north received a soaking rain followed by occasional hard showers during the day on Friday, September 9th.

Owing to the parched condition of the ground, no run-off occurred from the rain of Thursday night and only a little, except from areas within the city, from the day rain of Friday. The day rains of Friday were followed by a severe electrical storm, continuing from 6 P. M. to 9 P. M., during which the region north and northwest of the city was visited by intense rainfall which continued with reduced severity until 11.30 P. M.

Olmos Creek, north of the city, overflowed its banks and swept into the city about 10 P. M., reaching its crest within an hour. At the center of the city, the river, not more than 2 ft. above normal at 6 P. M., rose with increasing rapidity, overflowing its banks immediately north of the business district at midnight. Within an hour, a large part of the business quarter was inundated, and the crest was reached about 2 A. M. At this time, six of the principal north and south streets were channels carrying the swift flood waters across a great bend of the river. The water in these channels ranged in depth from 1 to 8 ft. where they crossed Houston Street, the principal east and west street and at places in the business area, the water was 12 ft. deep. North of Houston Street, the water spilled through five streets over a low divide to the west into the Upper San Pedro Basin.

In the central district, the river rose from 2 to 8 ft. above all bridge floors and at each bridge a great mass of débris was caught by the piers and railings.

In the northern part of the city, the rush of water in the meantime had inundated Breckenridge Park along the river and all the adjoining low residential and industrial areas. Hundreds of homes were flooded to depths varying from a few inches to 8 and 10 ft. In this section, a few poorly built

FIG. 3.—RIVER CHANNEL THROUGH HEART OF CITY.

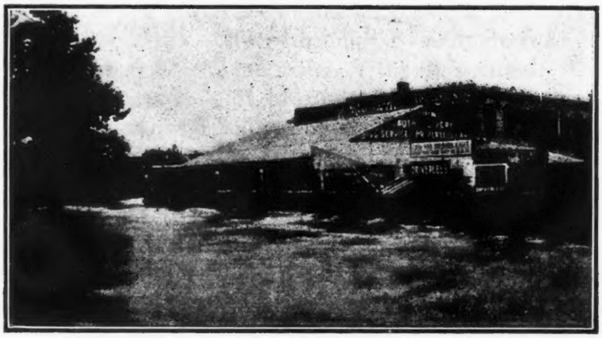


FIG. 4.—RIVER CHANNEL THROUGH HEART OF CITY, DURING FLOOD.

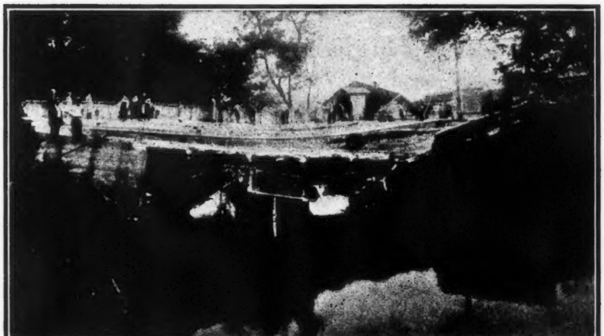


FIG. 5.—SOUTH ALAMO STREET BRIDGE.

frame dwellings were washed away. A scum of heavy fuel oil from some of the industries coated everything it touched, and, incidentally, everywhere below left clear and unmistakable high-water marks. In the business district, all the basements were filled, and most of the first floors were flooded to depths of from a few inches to 12 ft.

The severity of the flood crest in the business district was undoubtedly somewhat mitigated by the pondage of water and its retention by houses and other obstructions in the valley above. In like manner, the obstruction formed by the business district, the storage of water in basements, and probably, also, a considerable absorption from the river channel into the gravel beds which underlie many parts of the valley, reduced the flood markedly below. This effect was partly counteracted as far as the flooded residence district just below the business section was concerned, due to two low dams and the blocking of two railway trestles with drift, all of which were situated about 1 mile below the center of the city. Below these obstructions, the flood was carried by the river channel and natural overflow bottoms which have not been encroached on by retaining walls and buildings to the same extent as in the central and upper portions of the valley. Notwithstanding many narrow escapes there was practically no loss of life in the valley of the San Antonio River.

Owing to the comparatively light rainfall in the city, the small Upper San Pedro Basin did not flood badly, and its crest passed before the overflow from the San Antonio River. The latter inundated a number of buildings in the western edge of the business district, the water in it and the San Pedro forming a continuous sheet.

Alazan Creek, with a comparatively steep grade from the high ground northwest of the city, and its two branches flooding simultaneously, swept suddenly through the thickly settled low areas along its banks, playing havoc with bridges and tearing loose and destroying many cheap frame houses. It was in this section that the principal loss of life occurred. Conditions on Apache Creek were similar, but there was less development along its banks and the loss was smaller. Below the confluence of Alazan and Apache Creeks, the San Pedro again passes through a thickly settled area where there was considerable damage and loss of life. The total certain loss of life was 52, with 23 missing, most of whom were probably lost, making a total of 75.

The aggregate property damage has not been ascertained, but it was between \$5 000 000 and \$10 000 000. The greatest damage was to merchandise in flooded basements and on first floors. Next in amount was the loss in homes to clothing, furniture, and to the finish of walls and interiors from the muddy, oil-laden waters. The loss to the municipality was heavy, principally to pavements and wooden bridges destroyed or injured, together with the expense of sanitary measures and for cleaning up débris. One three-span concrete girder was destroyed by scour.

In general, substantial buildings were little damaged structurally, although there was a heavy loss of plate glass broken by water pressure and by drift, and, in total, a heavy damage for refinishing. The public service corporations suffered some loss, the principal item, however, being in decreased earnings for several days.

An interesting feature was the continuous low-pressure water service to the greater part of the city by the direct head of the artesian wells. An incident of engineering interest was the loss of practically all flooded wood block pavements which simply floated off. A large percentage of these blocks have been gathered up, however, and will be relaid.

Several concrete basement floors were burst upward by pressure from water which traveled underneath, through gravel beds, before the basements were flooded from the top. On the first floor of buildings, tile floors, in several instances, were cracked loose at the joints, and the first-floor concrete slabs were undoubtedly severely strained by pressure from beneath. This occurred where the basements were flooded and the water in the streets stood from 2 to 3 ft. higher than inside the closed doors where the occupants were attempting to prevent its entrance.

Another item that might be mentioned was the voyage of the two sea lions from the Municipal Zoo, one of which after visiting several porches on the morning after the flood, was captured 2 miles from home. The other is still reported to be at large about 75 miles down the river.

The rain storm which caused the San Antonio flood was typical in that it broke on the margin of the Edwards Plateau. Rainfall data over the water-sheds above San Antonio have been secured by the writer and by the Engineer Corps of the Army. These data are based on records at about twenty points; about one-third were secured by standard rain gauges and one-fourth by measurements in straight sided cans taken by farmers who keep more or less regular records. The remaining readings were secured by reducing measurements made in various classes of receptacles.

The testimony is so general as to the severity of the rain along the rocky rim of the Olmos water-shed and farther north in the plateau that there can be no question that at points the total precipitation was in excess of 20 in. Fig. 2 shows the total rainfall for the entire storm, from 6 P. M., Thursday, to midnight on Friday, a period of 30 hours. For the Olmos water-shed, the average rainfall was 14 in. and for that of San Antonio proper, below the city limits, it was 7.5 in., a mean depth of 12.5 in. for the entire basin.

The general storm over Central Texas was heavy, not only above San Antonio, but also at San Marcos and Austin, along the edge of the plateau. It is significant, however, that the heaviest rain, 19.5 in. in 12 hours or 23.1 in. in 24 hours, fell at Taylor, which is neither close to the coast nor adjacent to the plateau, but well out in the lower plain where previous statistics indicated that all the probabilities were against a rain exceeding 10 in. in 12 hours or 14 in. in 24 hours.

The available rainfall records of the general storm have been indicated on Fig. 1.

The greatest hourly intensities in the San Antonio storm were between 6 P. M. and 11.30 P. M. During this 5.5 hours, the rainfall ranged from 1.56 in. at the San Antonio Weather Bureau to 3.5 in., 4.5 in., 6.2 in., and possibly as high as 15 in., at various points. The average on the water-shed for this 5.5 hours has been placed at 6.5 in.

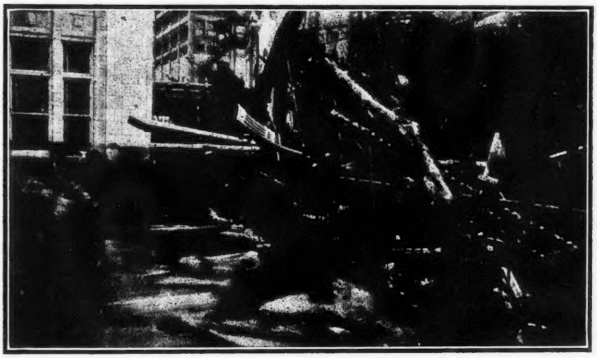


FIG. 6.—ST. MARY'S STREET BRIDGE.

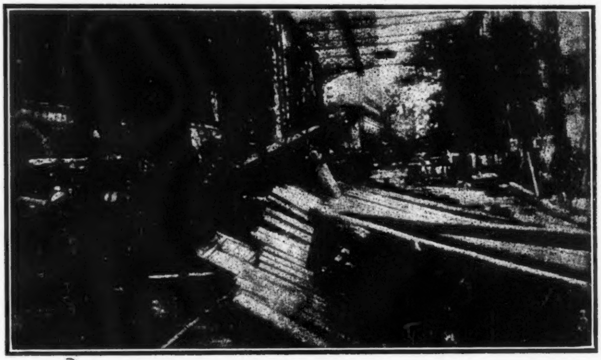


FIG. 7.—LOOKING NORTH ON NAVARRO STREET BRIDGE.

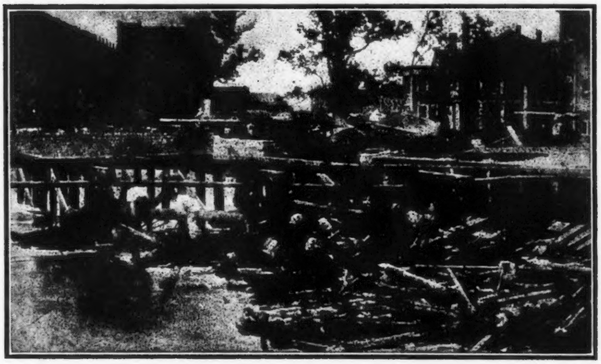


FIG. 8.—LOOKING TOWARD BUSINESS DISTRICT FROM PECAN STREET.

The Engineering Corps of the writer's firm has made a hydrographic survey, as careful as circumstances permitted, of the slopes and areas of the flood crest at various points. It appears that ½ mile above the city limits, the flood peak in the Olmos was between 31 000 sec-ft. as a minimum, and 39 000 sec-ft. as a maximum, from 32.4 sq. miles. This represents from 960 to 1 200 sec-ft. per square mile. The peak discharge at the center of the city is estimated at 23 700 sec-ft., or 580 sec-ft. per square mile from 41 sq. miles. Below the center of the city, 1½ miles, the crest is estimated at 1 500 sec-ft., or 333 sec-ft. per square mile from 45 sq. miles.

The rapid reduction in flood crest both in total and in second-feet per square mile is significant in that the reduction was very much more rapid with increased area than would be indicated by the usual curves for second-feet per square mile. This was doubtless due in part to the distribution of the rainfall, but the influence of the river-flood plain in impounding the flow was also a marked factor. The influence of buildings probably increased somewhat this retardation over the natural conditions of a timbered flood plain, although the streets were more efficient than natural channels. The average slope of the Olmos Valley above the city is 25 ft. per mile, the average fall of the river valley across the city being 14 ft. per mile.

One feature of the flood that will be difficult for Eastern engineers to understand is the comparatively small run-off. Above the city, the Olmos was not out of its banks more than 6 hours, the crest falling almost immediately. The flow increased from about 4 000 to 35 000 sec-ft. and fell again to 4 000 sec-ft. in 6 hours. At the center of the city, the flow ranged from 4 000 to 23 700 sec-ft. and back to 4 000 sec-ft., in about 11 hours.

From unofficial gaugings taken by the U. S. Geological Survey, on the day following the flood, and by gaugings of the water-stage register, and from other data, a rough computation has been made of the run-off for the storm for a period of 7 days, which indicates that it did not exceed 10 000 acre-ft. from an area of 42 sq. miles above the station. All the storm flow had practically run off within the period of one week. It is unfortunate that the water-stage register of the Geological Survey was located at the South Alamo Street concrete bridge which failed. Although the gauge itself was undamaged and recorded the levels except for 3 hours at the crest, the failure of the bridge interfered with the channel so that no accurate rating curve can be established for the flood conditions.

This estimated run-off of 10 000 acre-ft. is 36.4% of the depth of rainfall for the entire storm. However, by far the greater portion of this run-off was from 32.4 sq. miles above the city. It is probable, in fact, that the run-off was as great from the upper basin as from the entire basin, the absorption of flood-waters by gravel beds in the lower valley probably amounting to as much as the run-off from the lighter rain on the lower 9 sq. miles within the city.

Assuming that the run-off from the upper basin of 32.4 sq. miles was as great as at South Alamo Street, 5 miles down stream, that is, 10 000 acre-ft., according to the estimate, the run-off from the Olmos basin was about 42 per cent. Regarding the Thursday night and Friday rains, amounting to an aver-

age of 6.5 in., as producing only 10% of the actual run-off, there would have occurred an average run-off of not more than 70% from the 5.5-hour rain of Friday night falling on presaturated ground and amounting to from 3.5 to 15 in.

The flood peak on the Upper San Pedro Creek was not excessive either from the run-off of the earlier local storm, nor from the later overflow from the river. The maximum discharge was possibly 1 500 sec-ft. from 2 sq. miles.

On the Alazan Creek the water-shed of which is ideal for a quick flood concentration and on which a small earth dam was washed out, the peak flow was approximately 33 000 sec-ft. from 16.9 sq. miles, or at a rate of 1 950 sec-ft. per square mile.

The Apache Creek is estimated to have flowed at a rate of 15 500 sec-ft. from 22 sq. miles, or 704 sec-ft. per square mile.

The average rainfall on the water-sheds of these two creeks was probably as high as that on the Olmos Basin and greater than that for the combined Olmos-San Antonio Basins. The flood peaks from the creeks reached the San Antonio River before its own crest so that although the flood experienced in the lower river was of unusual proportions, it was not of the extreme character suffered in the individual streams above their junction near the southern city limits.

San Antonio has been subjected to floods from its river and the western creeks at intervals of from 8 to 15 years. The last damaging floods occurred, twice in 1913 and once in 1914. Spanish records tell of a flood in July, 1819, which was undoubtedly somewhat greater than the recent one. As a result of the floods of 1913-14, the city engaged Leonard Metcalf and H. P. Eddy, Members, Am. Soc. C. E., to report on methods of flood prevention, and their report filed in December, 1920, foretold the possibility of just such a flood as has happened.

As a result of the recent flood, a committee of local citizens and engineers has been formed to study the conditions and make immediate recommendations on flood prevention. It is probable that extensive works will be carried out in the near future.

The writer wishes to express his appreciation of valuable information secured from data collected in the report by Messrs. Metcalf and Eddy, to C. E. Ellsworth, Assoc. M. Am. Soc. C. E., of the U. S. Geological Survey, for advance stream records, and to Col. Edgar Jadwin, Corps of Engineers, U. S. A., for rainfall data and the opportunity of comparing discharge estimates.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

BUCKLING OF ELASTIC STRUCTURES

By H. M. Westergaard,* Esq.

Synopsis

The buckling of a slender column under an axial load and the collapse of a thin cylinder under a uniform external pressure have long been recognized to be related structural phenomena. In both cases the structural action may be described by the common term, buckling. If the column is perfectly straight, the load on it exactly central, and the material perfectly homogeneous and elastic, when the load is increased gradually from zero, the column remains straight until the load reaches a critical value at which the column bends out suddenly. In the same way, if the cylinder is exactly circular, the outside pressure on it absolutely uniform, and the material perfectly homogeneous and elastic, when the pressure is increased gradually from zero, the cylinder remains circular until the pressure reaches a critical value at which the cylinder deflects from the circular shape. In both cases, the deflections increase at the critical value of the load without any increase of the load. Until the deflections have become large, there is a neutral equilibrium, that is, one maintained at constant load throughout a continuous range of configurations. This neutral equilibrium is the criterion of what may be termed pure elastic buckling.

If the column is not perfectly straight, if the axial load on it is slightly eccentric, or if there are transverse loads in addition to the axial load, and, in the same way, if the cylinder is not perfectly circular and homogeneous, or the load on it not exactly uniform, then certain elements of the pure buckling may still be traced, and the structural action may still be characterized as buckling. The deflections increase slowly at first, with slowly increasing loads, but they increase rapidly as the loads approach the critical values. This kind of buckling may be looked on as a mixed action lying between two extremes, one of which is the pure elastic buckling first mentioned and the other the ordinary structural action in which the deformations and stresses are proportional to the loads, for example, the ordinary bending action of a beam loaded by transverse forces only.

Note.—These papers are issued before the date set for presentation and discussion. Correspondence is invited from those who cannot be present at the meeting, and may be sent by mail to the Secretary. Discussion, either oral or written, will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

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There is, as may be seen from the Bibliography (Part VII), a rather voluminous literature on buckling. Most of the analyses made in the past deal primarily, however, with the various cases of pure buckling and leave the questions of mixed buckling on the whole unsettled. The present investigation deals primarily with the subject of mixed buckling. A general analysis is made of this type of buckling, and the results are applied to various specific cases.

The method followed, to some extent, is to present results first, and theory afterward. For example, the formulas in Parts II and III are derived from the general formulas in Parts IV and V. The subject-matter is arranged as follows:

I.—Introduction (Articles 1-2), in which certain limiting assumptions are introduced, examples of buckling are discussed, and some terms are defined.

II.—Formulas Applying to Columns Which Are Loaded by Axial and by Transverse Forces at the Same Time (Articles 3-9), in which approximate and exact formulas are given for columns with hinged or fixed ends, loaded by a distributed or by a concentrated transverse load or by end couples.

III.—Slabs and Similar Structures (Articles 10-11), in which formulas are given for rectangular homogeneous slabs, simply supported on four sides, and for a double system of crossing beams connected at the cross-points.

IV.—General Theory of Structural Actions in Which the Stresses Are Not Proportional to the Loads, Although Hooke's Law Applies in Each Element of the Structure, and Although the Deformations Are Small (Articles 12-27), in which by the principle of energy minimum some general formulas are derived by which one may compute any structural effect, such as bending moment, stress, or deflection, in mixed buckling, provided the corresponding case of pure buckling is known.

V.—Differential Equations Applying to Some Definite Cases (Articles 28-31).

VI.—Summary (Article 32).

VII.—Classified Bibliography.

I.—Introduction

1.—Limiting Assumptions, Examples of Buckling, and Definitions of the Terms Orthostatic, Astatic, and Heterostatic.—Two assumptions are usually made in the analysis of stresses in structures, which have been shown by experience, by test, and by analysis, to be warranted within a wide range of cases, and they will be made throughout this investigation. One assumption is that at all points the unit deformations, or strains, are proportional to or are linear functions of the stresses; the other is that the elastic deflections from the original shape are small and may be considered as infinitesimal when compared with the main dimensions of the structure, or, in this sense, the structures are assumed to be stiff. These two assumptions, of elasticity and stiffness, limit and simplify the problem to be investigated.

When both assumptions are warranted, one may conclude in many, though not in all, cases that a stress or a deflection produced by two loads combined is the sum of the stresses or deflections produced by each of the two loads acting separately; that is, the principle of superposition is found to apply to the loads, stresses, and deflections. The general principle of superposition, when it applies, states that the effect of two or more causes combined is the sum of the effects of the separate causes; for example, the stress due to a total load consisting of live load plus dead load is computed, in accordance with this principle, by adding the live load stress to the dead load stress. An important conclusion drawn from the law of superposition of loads, stresses, and deflections is that when all the external loads are increased in a certain proportion, the stresses and the deflections are increased in the same proportion.

It is evident that the law of superposition does not apply generally to the cases of buckling which have been mentioned. In fact, one may define elastic buckling in the most general sense as a structural action in which the stresses are not, in general, proportional to the loads, although Hooke's law applies in each element of the structure, and although the displacements are small; or, as a structural action in which the assumptions of elasticity and stiffness apply, but in which the law of superposition of loads, stresses, and deflections does not apply to the structure as a whole, although it may apply to certain parts of it. It is intended to investigate elastic buckling in this sense of the term.

A special nomenclature for the two actions which have been referred to as pure buckling and mixed buckling will be introduced later. First, the nature of these actions will be explained by means of examples.

Beginning with pure buckling: The example, already referred to, of the straight slender column buckling under a central axial load, shows the most characteristic features. Let the ends be simply supported or hinged, and the cross-section be constant. Then, at the critical load, given by Euler's well-known formula, the transverse deflections which were zero or negligible at smaller loads, increase suddenly. The neutral equilibrium, maintained at this constant critical load throughout a continuous range of increasing deflections, is the criterion of pure buckling. The load may be increased beyond the critical value at which the bending ordinarily begins, and still the column may be made to remain in equilibrium in an undeflected state. This equilibrium is unstable, however; that is, it would be disturbed by a small force from the side, or by a small local eccentricity of the elastic resistances of the cross-section. At higher "critical loads", other states of neutral elastic equilibrium may be obtained, in each of which the column may buckle into a curve consisting of some definite number of half waves. All the neutral equilibria produced under the higher critical loads are unstable. When they do occur, they will tend to revert to the neutral equilibrium which was produced first, and the maximum load which can be applied will drop to the corresponding lowest critical value. The lowest critical load at which buckling first occurs is of particular interest, because it defines a limit of the strength, but it will also be shown, subsequently, that the study of the higher critical loads and of

their corresponding neutral equilibria leads to a solution of important structural problems.

Fig. 1 illustrates some typical cases of pure buckling of elastic structures. Fig. 1 (*A-F*) shows some originally straight columns with various end conditions. Column *A* has hinged ends, Column *B* fixed ends, Columns *C* and *D* have one free end, while Columns *E* and *F* have elastic supports at the ends. In the case of Column *C*, the load at the free end is assumed to remain parallel to itself while the deflections increase, while in Column *D* the load applied at the upper end of the column is inclined in proportion to the deflection of that point. If Columns *C* and *D* are of the same dimensions and material, then Column *D* will buckle at a lower load than Column *C*. In the cases of Columns *E* and *F*, the same types of buckling may occur as in the case of Column *A*, but, in addition, neutral equilibria may be produced by the deformation of the spring supports, whereby the column moves into a position which is inclined with respect to the vertical, as indicated in the diagram. Fig. 1 (*G*) shows a section of a long, thin-shell cylinder under uniform outside pressure. When the pressure reaches a critical value, the surface buckles, as indicated by the dotted line. The typical neutral equilibrium is found here also. Furthermore, a larger number of waves in the deflected surface might be formed at higher "critical pressures". Only the lowest critical load gives a stable neutral equilibrium. All the neutral equilibria which could be maintained at the higher critical pressures, are unstable; that is, the general characteristic features mentioned for a slender, straight column exist also in this case.

Fig. 1 (*H*) shows a curved compression member which carries forces at the ends and, in addition, a distributed pressure from the convex side. If the original center line is a possible line of pressure, that is, if it is a funicular curve for the side pressures, then buckling may take place, as in the case of the circular cylinder. Fig. 1 (*J*) shows a bridge of the pony-truss type; the sidewise buckling of the arch is restrained by the stiffness of the verticals, but this restraint may be overcome at a certain limiting pressure in the arch. Fig. 1 (*K*) and (*L*) represent slabs loaded from the ends; the case shown in Fig. 1 (*L*) occurs, for example, in the web of a plate girder under the influence of the shear. The edges may be either fixed or free to turn. At critical loads, the slabs will buckle into various surfaces. The case shown in Fig. 1 (*M*) is similar to that in Fig. 1 (*K*), only the continuous slab is replaced by a double system of crossing beams. Fig. 1 (*N*) is a hinged-end column with a continuous elastic support from the side; the springs shown in the diagram are assumed to be without stress when the column is straight. The possible curves of deflection are similar to those in Fig. 1 (*A*), but the critical loads, if the column itself is the same, are essentially higher, and the number of half waves of the deflected curve corresponding to the lowest load is likely to be two or more, depending on the stiffness of the side support. Fig. 1 (*O*) shows a cylinder under outside pressure, but with the surface stiffened by rings. The relation of this case to that of the cylinder, Fig. 1 (*G*), is similar to the relation existing between the columns, Fig. 1 (*N*) and (*A*). If the two cylinders are similar, Fig. 1 (*O*) will collapse at a higher load than Fig. 1 (*G*), and it is likely that a larger number of surface waves will be formed in the process of

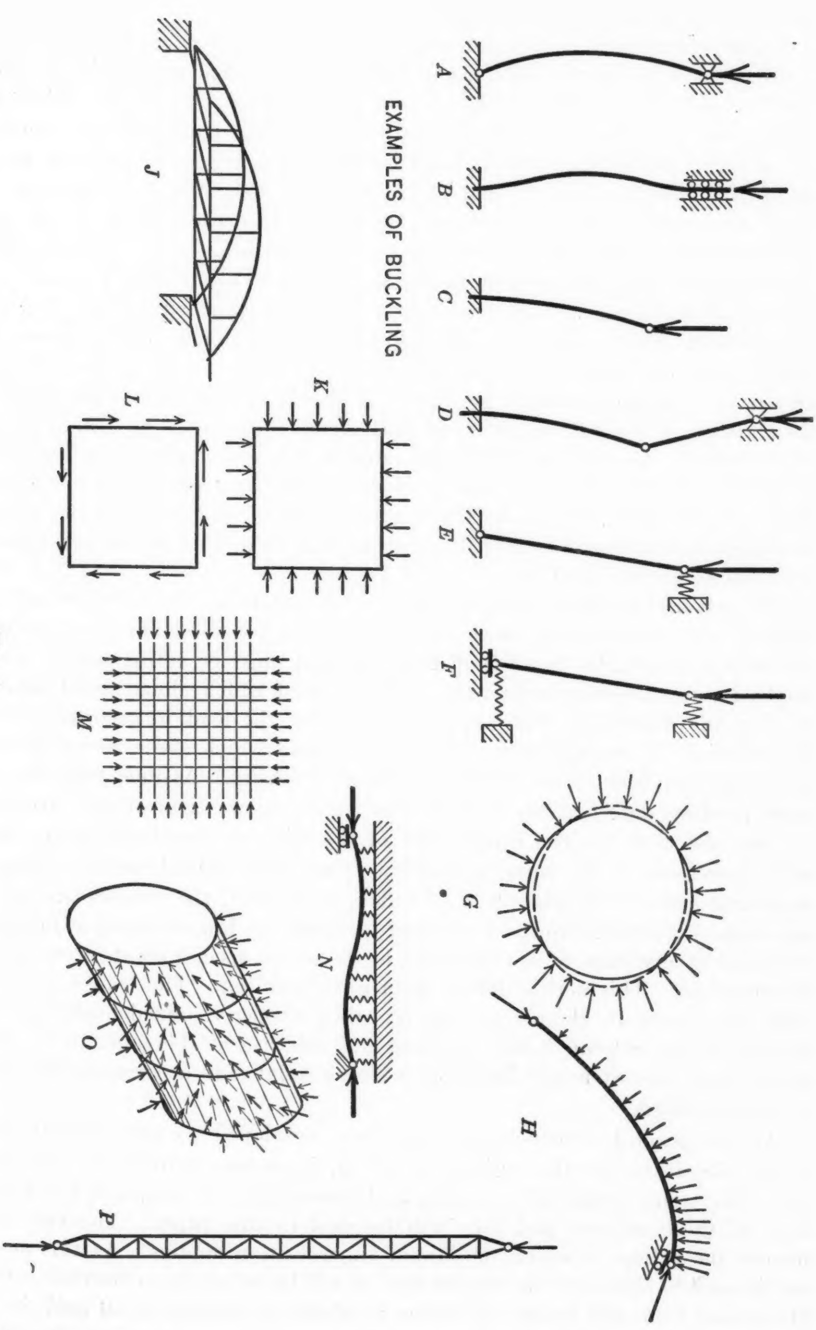


FIG. 1.

buckling. Finally, Fig. 1 (P) shows a truss column, which may buckle in the same way as the column shown in Fig. 1 (A).

Other examples may be mentioned: A long shaft or wire may buckle into a spiral shape under the influence of either torsion alone or torsion combined with an axial load. This action is most easily demonstrated by twisting a short piece of string. A high narrow beam or blade may bend out sidewise, tip, or twist, under the influence of a bending load which lies initially in the plane of maximum rigidity, that is, in the plane of symmetry containing the large dimension of the cross-section. At a certain critical value of the load, the torsional rigidity becomes insufficient, and the beam tips. This action is demonstrated when one tries to hold a long strip of paper or a T-square as a horizontal cantilever with the flat side vertical. A thin cylindrical shell or a tubular strut may buckle into a double-curved surface under the influence of a compressive load parallel to the axis. Buckling may occur in a spherical shell loaded by a uniform outside pressure. A closely related dynamical action is the whirling of a shaft, whereby the shaft bends out when a critical speed has been reached. Concerning the larger number of cases of pure buckling which have been mentioned, reference is made to the literature in the classified Bibliography (Part VII). As far as pure buckling is concerned, the present investigation deals mainly with the general aspects of this action and with its relations to mixed buckling.

The nature of mixed buckling will now be explained. Mixed buckling may occur in any structure in which pure buckling is possible; for example, with any of the structures shown in Fig. 1. A load must be acting which can be resolved into two component parts: One, a load which alone would produce stresses and deflections that are proportional to the load in accordance with the principle of superposition; the other, a load which alone would produce pure buckling when increased to a critical intensity. Assume that the two loads produce deformations and stresses in the same parts of the structure. Assume also that the two components of the load act together and are gradually increased in the same proportion from small initial values. Then, a structural action will take place in which, in general, the stresses are not at any stage proportional to the loads, and in which the law of superposition does not apply at any stage except, possibly, to stresses at special points or to special deformations. This action differs from pure buckling. The load is a mixed load, and, therefore, the action may be called, properly, mixed buckling. An example is the column loaded endwise and sidewise at the same time. Evidently, each case of mixed buckling is closely related to a corresponding case of pure buckling.

As no general nomenclature has been developed to the present time in the literature for the actions described, it seems desirable to introduce one. The terms, orthostatic, astatic, and heterostatic are proposed for discussions of these actions, and they will be used in this paper. The first term denotes the action in which the law of superposition applies. As this action may be said to represent the regular case, it will be called the orthostatic action. The second term will denote the action in which an increase of all loads in the same proportion to critical values produces pure buckling, characterized by a

neutral elastic equilibrium. Such action will be called astatic action. This term is explained by the common usage of the word "astatic" as indicating certain conditions under which a neutral equilibrium is created. The neutral equilibrium is sometimes called an astatic equilibrium, a usage which will be followed by the writer. With the terms introduced, it may be said, for example, that a "neutral or astatic equilibrium characterizes a pure buckling and occurs at certain stages of an astatic action." The third term denotes what has been described as mixed buckling. Such action will be called heterostatic action. Loads producing orthostatic, astatic, and heterostatic actions, respectively, will be referred to as orthostatic, astatic, and heterostatic loads. A heterostatic load, therefore, can be resolved into two components: One an orthostatic load, the other an astatic load. From what has been stated previously, it follows that this investigation will deal mainly with general aspects of astatic action, and with heterostatic action. It will be shown that the cases of heterostatic action can be solved, by certain general formulas, in terms of corresponding cases of astatic and orthostatic actions.

2.—Remarks Concerning the Bibliography.—The classified Bibliography (Part VII) is not intended to include, in general, references dealing exclusively with straight columns which have constant cross-sections and which carry axial end loads only. This case is treated more or less extensively in almost any textbook on Mechanics of Materials. It is sufficient for the present purpose to point to Euler's analysis of columns, which dates back to 1757, and to the progress which has been marked by the introduction of such formulas as that of Rankine, the parabolic, and the straight-line formulas. A large amount of experimental work has been completed in this field. The deviations from Euler's formula are usually explained by the inevitable presence of small initial eccentricities. In fact, some of the textbooks* interpret the formulas mentioned as approximate representations of the secant formula, which applies to eccentric loading and which may be used when some definite "equivalent initial eccentricity" has been introduced.

The comparatively recent work by Kármán on straight columns is quoted in the Bibliography (Part VII, Section *B*). His experiments and analysis are noteworthy on account of the light they throw on the influence of stresses above the proportional limit. This matter is important in the analysis of the shorter columns. Especial attention is called to the treatment of S. Timoshenko of a great variety of cases of pure elastic buckling (Bibliography, Part VII, Section *A*), and also to the analysis of cylindrical shells by Goupil (Section *G*), in which a special case of the general formula for heterostatic action is derived.

II.—Formulas Applying to Columns Which Are Loaded by Axial and by Transverse Forces at the Same Time.

3.—Examples of Applications, Notation, and Current Approximate Formulas.—Straight columns loaded axially and at the same time bent by transverse forces are found in many structures. Examples of such columns are as fol-

* See, for instance, J. E. Boyd, "Strength of Materials," Ed. 1917.

lows: Vertical columns which are loaded sidewise by wind pressure, the latter being transferred to the column either as concentrated forces or as a distributed load; horizontal or inclined compression members which are bent by their own weight; struts in aeroplanes which are bent by wind pressure; and the connecting rod in an engine which is bent by a dynamical action equivalent to a distributed load.

In this Article, and in the subsequent articles, the following notation will be used:

Let l = length of column;

EI = modulus of elasticity times moment of inertia of cross-section;

P = actual axial load;

Q = lowest critical axial load at which buckling would occur. When EI is constant throughout the length, Euler's formula gives: For columns with hinged ends, $Q = \pi^2 \frac{EI}{l^2}$; and for columns with fixed ends, $Q = 4\,\pi^2 \frac{EI}{l^2}$;

M' = resultant bending moment due to combined pure buckling action and bending, that is, to the combined axial and transverse loads. (M' is the moment in the "mixed" or "heterostatic" action);

M = bending moment when the transverse load acts alone (the moment in the case of orthostatic action).

Indices c and e are used to designate the moments at the center and at the ends, respectively. For instance, M_c = the moment at the center due to the transverse load alone. α, β, and μ are constants entering into the current approximate formulas. Further notation will be introduced in Article 7, page 468.

Two current approximate formulas will be mentioned. In the first the ratio $\frac{\beta}{\alpha}$ enters. This ratio depends on the distribution of the load and on the end conditions. For example, columns with hinged ends and with a uniform transverse load have $\frac{\beta}{\alpha} = 9.6$; columns with hinged ends and with a concentrated load at the center have $\frac{\beta}{\alpha} = 12$; usually, $\frac{\beta}{\alpha}$ is not very far from 10. With the notation previously given, the formula may be written:*

$$M' = \frac{M}{1 - \frac{\alpha}{\beta}\frac{P\,l^2}{E\,I}} \quad \text{(1)}$$

In the other approximate formula, the constant, μ, occurs, which is stated as being not very far from unity. Sometimes, $\mu = 1$. The formula† is

$$M' = M\,\frac{\mu\,Q}{\mu\,Q - P} \quad \text{(2a)}$$

* M. Merriman, "Mechanics of Materials," 11th Edition (1915), p. 256.

† A. Ostenfeld, "Teknisk Elasticitetslære", 3d Edition (1916), p. 456.

or, when $\mu = 1$,

$$M' = M \frac{Q}{Q - P} \quad \text{.......................} \quad (2b)$$

When $\frac{\beta}{\alpha} = \mu \pi^2$, and when $Q = \frac{\pi^2 E I}{l^2}$, Formulas (1) and (2*a*) give identical results. These formulas furnish good approximations in many, but not all, cases.

4.—Proposed Design Formulas for Straight Columns with Uniform Cross-Section.—The theory leading to the formulas proposed here will be discussed later. The exact results are obtained as infinite series, which, however, converge so rapidly that for almost all practical purposes it is sufficient to include only the first two terms of each series. In formulas for design purposes, it is also permissible to simplify the constants entering into them by omitting the less significant decimals.

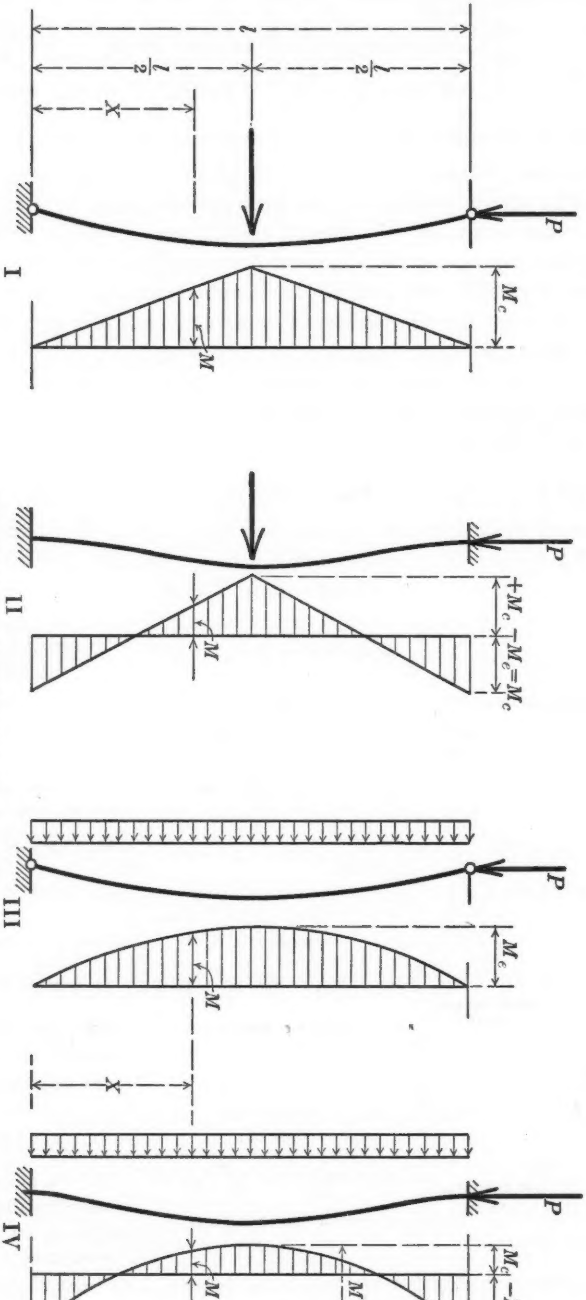
Four cases which have a particular interest, are shown in Fig. 2. The corresponding proposed design Formulas (3), (3*a*), (4), (4*a*), (5) and (6), are given in Table 1.

TABLE 1.—PROPOSED DESIGN FORMULAS FOR STRAIGHT COLUMNS.

EI is assumed to be constant throughout the length. The limiting errors are for positive P (compression). Cases I, II, III, and IV are illustrated in Fig. 2.

		Concentrated load at center.	Uniformly distributed load.
Hinged-end columns: Moments at center, $Q = \frac{\pi^2 E I}{l^2}$.		Case I: $M_c' = M_c \left(1 + 0.8 \frac{P}{Q - P}\right) \text{..} (3)$ (error less than 1.4%).	Case III: $M_c' = M_c\left(1 + 1.03 \frac{P}{Q - P}\right) \text{...} (4)$ (error less than 0.2%) or, approximately, $M_c' = M_e \frac{Q}{Q - P} \text{..........} (4a)$ (error less than 3.2%).
Fixed-end columns; $Q = \frac{4\pi^2 E I}{l^2}$.	Moments at center.	Case II: $M_c' = M_c \left(1 + 0.8 \frac{P}{Q - P}\right) \text{..} (3)$ (error less than 1.4%).	Case IV: $M_c' = M_c\left(1 + 1.2 \frac{P}{Q - P}\right) \text{....} (5)$ (error less than 1.3%, and on the safe side; when $\frac{P}{Q} \leqq 0.9$, the error does not exceed 1%).
	Moments at ends.	Case II: $M_e' = M_e\left(1 + 0.8 \frac{P}{Q - P}\right) \text{.} (3a)$ (error less than 1.4%).	Case IV: $M'_e = M_e \left(1 + 0.65 \frac{P}{Q - P}\right) \text{...} (6)$ (error less than 6.5% and on the safe side; for $\frac{P}{Q} \leqq 0.5$, the error is less than 1%).

If a concentrated load and a distributed transverse load act at the same time, their combined effect can be computed by adding the values given by



COLUMNS WITH TRANSVERSE LOADS
FIG. 2.

the two separate formulas (in Table 1) which apply to these two separate loads; in other words, although the law of superposition does not apply to the total load, consisting of axial loads plus transverse loads, it does apply to the transverse loads alone as long as the axial load retains one constant value.

The formulas also apply, approximately, when P is negative, that is, when the axial load is a tension. In that case a catenary action takes place in which the axial load causes a reduction of the bending moments.

5.—Exact Formulas from Which the Preceding Design Formulas Have Been Derived.—Comparison with the Approximate Formulas.—Formulas (3) to (6), in Table 1, have been derived by simplification of exact expressions given as infinite series. These expressions apply both when P is positive, representing a compression, and when P is negative, representing a tension. P must not be equal to any of the critical loads.

In Cases I and II (Formulas (3) and (3*a*), for $M'_{c \text{ or } e}$ and $M_{c \text{ or } e}$), the column is loaded transversely at the center, and has fixed or hinged ends. The exact formula for the bending moments at the center in Cases I and II, and at the ends in Case II, is:

$$M'_{c \text{ or } e} = M_{c \text{ or } e}\left[1 + \frac{8}{\pi^2}\frac{P}{Q-P} + \frac{8}{\pi^2}\frac{1}{3^2}\frac{P}{9\,Q-P} + \frac{8}{\pi^2}\frac{1}{5^2}\frac{P}{25\,Q-P} + \cdots\cdots + \frac{8}{\pi^2}\frac{1}{n^2}\frac{P}{n^2\,Q-P} + \cdots\cdots\right] = M_{c \text{ or } e}\left[1 + 0.811\frac{P}{Q-P} + 0.0901\frac{P}{9\,Q-P} + 0.0324\frac{P}{25\,Q-P} + \cdots\cdots\right] \quad \text{(7)}$$

in which $n = 1, 3, 5\ldots\ldots$

In Case III (Formulas (4) and (4*a*), for the moment at the center, M'_c), the column has hinged ends and carries a uniform transverse load:

$$M'_c = M_c\left[1 + \frac{32}{\pi^3}\frac{P}{Q-P} - \frac{32}{\pi^3}\frac{1}{3^3}\frac{P}{9\,Q-P} + \cdots\cdots \pm \frac{32}{\pi^3}\frac{1}{n^3}\frac{P}{n^2\,Q-P} \mp \cdots\cdots\right] = M_c\left[1 + 1.032\frac{P}{Q-P} - 0.0382\frac{P}{9\,Q-P} + 0.0083\frac{P}{25\,Q-P} - \cdots\cdots\right] \quad \text{(8)}$$

in which $n = 1, 3, 5\ \ldots\ldots$

Case IV (Formula (5)), gives the moment at the center, for a column with fixed end and uniform load:

$$M'_c = M_c\left[1 + \sum_{1,2,3,.}^{n} \frac{12\,(-1)^{n-1}}{\pi^2\,n^2}\frac{P}{n^2\,Q-P}\right]$$

$$= M_c\left[1 + 1.216\frac{P}{Q-P} - 0.304\frac{P}{4\,Q-P} + 0.1352\frac{P}{9\,Q-P} - 0.0760\frac{P}{16\,Q-P} + 0.0486\frac{P}{25\,Q-P}\cdots\right] \quad \text{(9)}$$

Case IV (Formula (6)), gives the moments at the ends for a fixed-end column, with uniform load:

$$M'_e = M_e \left[1 + \sum_{1,2,3\ldots}^{n} \sum_{1,2,3\ldots}^{m} \frac{6}{\pi^2 n^2} \frac{P}{n^2 Q - P} \right]$$

$$= M_e \left[1 + 0.608 \frac{P}{Q - P} + 0.1520 \frac{P}{4\,Q - P} + 0.0676 \frac{P}{9\,Q - P} \right.$$

$$\left. + 0.0380 \frac{P}{16\,Q - P} + 0.0243 \frac{P}{25\,Q - P} \ldots \right] \ldots\ldots\ldots (10)$$

The limiting errors indicated in Table 1 are found by comparison with the exact Formulas (7) to (10). Formula (6), derived from Formula (10), gives the greatest deviation. However, the error remains less than 1% when $\frac{P}{Q} \leqq 0.5$. A quite satisfactory approximation is obtained by using the first three terms in Formula (10), which may then be written in the simplified form:

$$M'_e = M_e \left[1 + 0.6 \frac{P}{Q - P} + 0.2 \frac{P}{4\,Q - P} \ldots \right] \ldots\ldots\ldots (10a)$$

Formulas (7) to (10) are rapidly converging series. For instance, $P = 0.5Q$, that is, the actual axial load equal to one-half of the critical axial load, gives the following values of $\frac{M'}{M}$, when two, three, or four, etc., terms of each series are included:

From Formula (7): 1.811, 1.816, 1.817, 1.817, ...
From Formula (8): 2.032, 2.030, 2.030, ...
From Formula (9): 2.216, 2.173, 2.181, 2,179, 2.180, ...
From Formula (10): 1.608, 1.630, 1.634, 1.635, 1.635, ...

The approximation obtained with the current approximate Formulas (1), (2a), and (2b), will now be examined in a typical case. Consider a hinged-end column loaded by a concentrated transverse force at the center. In that case, $\frac{\beta}{\alpha} = 12$. This value corresponds to $\mu = \frac{12}{\pi^2}$ in Formula (2a). The deviations from the exact values are then in percentage:

For $\frac{P}{Q}$ .... =	0.2	0.5	0.9	1.0 (limiting case):
Deviations by using Formulas (1) and (2a)........... =	— 1%	— 7%	— 54%	— 100%
Deviations by using Formula (2b) ... =	+ 4%	+ 10%	+ 20%	+ 23%

This comparison bears out the desirability of a revision of the current formulas. It must be stated, though, that, on account of the rapid variations

of M' when $\frac{P}{Q}$ approaches unity, the great deviations when $\frac{P}{Q} = 0.9$ or 1.0 are of comparatively less significance than the errors at smaller values of $\frac{P}{Q}$.

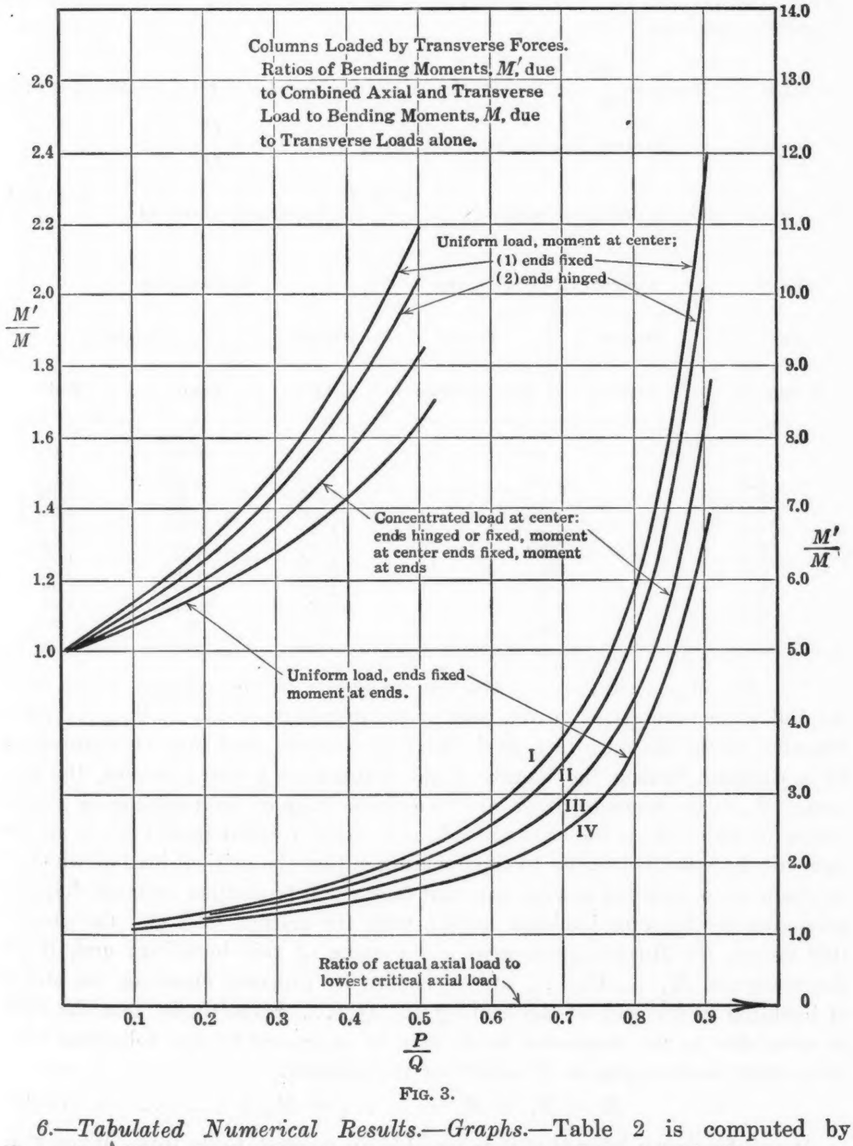


FIG. 3.

6.—Tabulated Numerical Results.—Graphs.—Table 2 is computed by Formulas (7) to (10). Fig. 3 represents the same data graphically.

7.—A More General Formula from Which the Preceding Formulas Have Been Derived.—The notation given in Article 3 is here supplemented, as follows:

$Q_1, Q_2, Q_3 \ldots Q_n \ldots$ = series of critical axial loads at which buckling (astatic elastic equilibrium) can take place. Q_1 is equal to Q in the preceding formulas. When both ends are fixed or both ends hinged, and when the cross-section is constant, then $Q_n = n^2 Q$. As mentioned in the Introduction, the neutral or astatic equilibria corresponding to the higher critical loads are unstable equilibria.

TABLE 2.—RATIOS, $\frac{M'}{M}$, OF TOTAL MOMENTS TO MOMENTS DUE TO TRANSVERSE LOADS ALONE FOR VARIOUS VALUES OF $\frac{P}{Q}$.

P = axial load ; for hinged ends, $Q = \frac{\pi^2 E I}{l^2}$; for fixed ends, $Q = \frac{4 \pi^2 E I}{l^2}$.

LOAD:	CONCENTRATED AT CENTER.		DISTRIBUTED.		
Ends:	Hinged.	Fixed.	Hinged.	Fixed.	
Moment at:	Center.	Ends or center.	Center.	Center.	Ends.
$\frac{P}{Q}$ = 0	1.000		1.000	1.000	1.000
0.1	1.091		1.114	1.129	1.073
0.2	1.205		1.257	1.290	1.163
0.3	1.351		1.442	1.502	1.277
0.4	1.546		1.686	1.782	1.428
0.5	1.817		2.030	2.18	1.636
0.6	2.223		2.545	2.78	1.95
0.7	2.901		3.405	3.78	2.46
0.8	4.25		5.13	5.79	3.48
0.9	8.31		10.29	11.87	6.52
1.0	∞		∞	∞	∞

$M_1, M_2, M_3 \ldots M_n \ldots$ = bending moments in the column which have the following properties: First, each of the moments, $M_1 \ldots\ldots M_n \ldots\ldots$, is a function of the distance measured along the column, and may be represented by a diagram having the length of the column as a base; second, the diagram, M_n, is the moment diagram at a certain stage of the buckling or astatic action in which Q_n is the critical load; it is easily verified that if M_n is such a moment diagram, belonging to the buckling action the critical load of which is Q_n, then M_n multiplied by any constant factor will be another moment diagram belonging to the same buckling action, with the critical load Q_n; the greater this factor, the further progressed is the stage of this buckling; and, third, the moments, $M_1 \ldots M_n \ldots$, are so chosen by properly choosing the stages of buckling under each of the loads $Q_1 \ldots Q_n \ldots$, respectively, that the total moment due to the transverse loads may be expressed by the following relation, which is to apply at all points of the column:

$$M = M_1 + M_2 + \ldots\ldots + M_n + \ldots\ldots\ldots\ldots \quad (11)$$

It will be shown later that it is possible, in general, to do this. When it is possible to establish the relation, Formula (11), it may be done by mutiplying any set of moment diagrams which are proportional to $M_1, M_2, \ldots M_n \ldots$ and which are produced in astatic actions, by a proper set of factors.

The more general formula from which Formulas (7) to (10) have been derived, will now be indicated. The derivation of the formula will follow later (Part IV). The formula has a wide field of application, but will appear in Part IV as a special case of a still more general formula which applies to any heterostatic action. The formula applies primarily to those cases of axially and transversally loaded straight columns in which the supports do not absorb any elastic energy, that is, it applies, for example, when the ends are fixed or simply supported (hinged), or when one end is free and the other end fixed, and also when there are inelastic supports at intermediate points. The formula also applies in general even when the supports are elastic, but in that case some qualifications must be introduced as to the make-up of the functions, $M_1 \ldots\ldots M_n \ldots\ldots$ This result will follow from the general theory in Part IV. The formula applies whether the cross-section is constant or varies, or whether the axial load is applied at the ends or at other points. The formula is:

$$M' = M + M_1 \frac{P}{Q_1 - P} + M_2 \frac{P}{Q_2 - P} + \ldots\ldots + M_n \frac{P}{Q_n - P} + \ldots\ldots \quad (12)$$

The essential difficulty in the use of Formula (12) lies in the determination of the functions, $M_1 \ldots\ldots M_n \ldots\ldots$ The writer will now show how these difficulties are overcome in various specific cases.

8.—Deriving the Specific Formulas from the General Formula.—The formulas applying to the four cases in Fig. 2 will now be derived from the general Formula (12). The cross-section is again assumed to be constant, and the same typical method will be seen to apply in all four cases.

Case I.—Hinged-end column, carrying a concentrated transverse load at the center (Fig. 2 (I)): The formula which is to be derived is Formula (7). The derivation of the approximate Formula (3) from Formula (7) was indicated in Article 5.

The first step is to find the deflections and the bending moments in the buckling actions in which Q_1, $Q_2 \ldots\ldots Q_n \ldots\ldots$ are the critical loads. To the critical load, Q_n, corresponds a buckling curve or curve of deflections consisting of n half waves. From the theory of columns, as given in texts on Mechanics of Materials, this curve is known to be a sine wave. The bending moments are found as the second derivatives of the deflections multiplied by the constant factor, $\pm EI$. It follows also that the moment diagram, M_n, will be shaped as a sine curve with n half waves. Let x be the distance measured along the column from the lower end; then the equation for M_n may be written:

$$M_n = C_n \sin\left(\frac{n \pi x}{l}\right) \ldots\ldots\ldots\ldots\ldots\ldots (13)$$

where C_n is a constant. The bending moment, M, due to the transverse load alone, should now be expressed in the form of the series, $M = \Sigma M_n$ (Formula (11)). M is given by the triangular moment diagram in Fig. 2 (I). The series, ΣM_n, is a trigonometric series. From the theory of Fourier series* it is known that any diagram can be represented by an infinite trigonometric series (Fourier series). It is possible, therefore, to find a set of constants, C_n,

* For instance, Byerly, "Harmonic Functions", 4th ed. (1906).

which, introduced into Formula (13), make $M = \Sigma\, M_n$. From the theory of Fourier series we have:

$$C_n = \frac{2}{l} \int_0^l M \sin\left(\frac{n\,\pi\,x}{l}\right) d\,x \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Substituting the values for M given in Fig. 2 (I), integrating, and forming the expressions for M_n, we find:

$$\text{for } n \text{ even, } M_n = 0$$

$$\text{for } n \text{ uneven, } M_n = (-1)^{\frac{n-1}{2}} \frac{8\,M_c}{\pi^2\,n^2} \sin \frac{n\,\pi\,x}{l}.$$

Hence, the series for M may be written:

$$M = \Sigma\, M_n = M_c \left[\frac{8}{\pi^2} \sin \frac{\pi\,x}{l} - \frac{8}{\pi^2} \frac{1}{3^2} \sin \frac{3\,\pi\,x}{l} + \frac{8}{\pi^2} \frac{1}{5^2} \sin \frac{5\,\pi\,x}{l} - \ldots\ldots \right]$$

The expressions for M_n appear in this series with the constant factor M_c. By substituting these expressions for M_n in Formula (12) and by putting $Q_n = n^2 Q$, the following formula is found for the moment, M', produced at any point, under the combined influence of the transverse and the axial load:

$$M' = M + M_c \left[\frac{8}{\pi^2} \frac{P}{Q - P} \sin \frac{\pi\,x}{l} - \frac{8}{\pi^2} \frac{1}{3^2} \frac{P}{9\,Q - P} \sin \frac{3\,\pi\,x}{l} \right.$$

$$\left. + \frac{8}{\pi^2} \frac{1}{5^2} \frac{P}{25\,Q - P} \sin \frac{5\,\pi\,x}{l} - \ldots\ldots\ldots\ldots \right]$$

We are particularly interested in the maximum moment, which occurs at the center. At that point, n uneven gives $\sin \dfrac{n\,\pi\,x}{l} = \pm 1$. The substitution of these values, ± 1, in the foregoing equation for M' leads directly to Formula (7) which was to be derived.

The other cases, II, III, and IV, in Fig. 2, for which special formulas have been given, can be treated in a similar manner. In all cases the moment diagrams, M_n, in pure buckling action (astatic action) are trigonometric diagrams, and the series, $\Sigma\, M_n$, are Fourier series which represent the triangular and parabolic moment diagrams given in Fig. 2. Each of these cases will now be discussed.

Case II.—Column with fixed ends, and with a transverse load at the center: In the deflected curve, the quarter points will be points of inflection. The column will be in equilibrium when the middle-half is bent according to the laws applying to the whole length in Case I, and the outer quarters of the length will be bent into the same shape as the middle quarters. It follows that Formula (7) applies also to this case and expresses both the moments at the ends and at the center.

Case III.—Hinged ends, uniform transverse load: Formulas (13) and (14) apply also to this case, only, the values of M are to be taken from the parabolic moment diagram. M_n is found to be:

$$\text{for } n \text{ even, } M_n = 0$$

$$\text{for } n \text{ uneven, } M_n = \frac{32\,M_c}{\pi^3\,n^3} \sin \frac{n\,\pi\,x}{l}$$

The substitution of these values, with $x = \frac{l}{2}$, in the general Formula (12), leads to Formula (8).

Case IV.—Fixed ends, uniform transverse load: When the distance, x, is measured from the center, we find:

$$M = M_0 \left(\frac{1}{3} - \frac{4\,x^2}{l^2}\right)$$

where,

$$M_0 = 3\,M_c = -\,1.5\,M_e$$

M_n takes the form:

$$M_n = C_n \cos n \left(\frac{2\,\pi\,x}{l}\right)$$

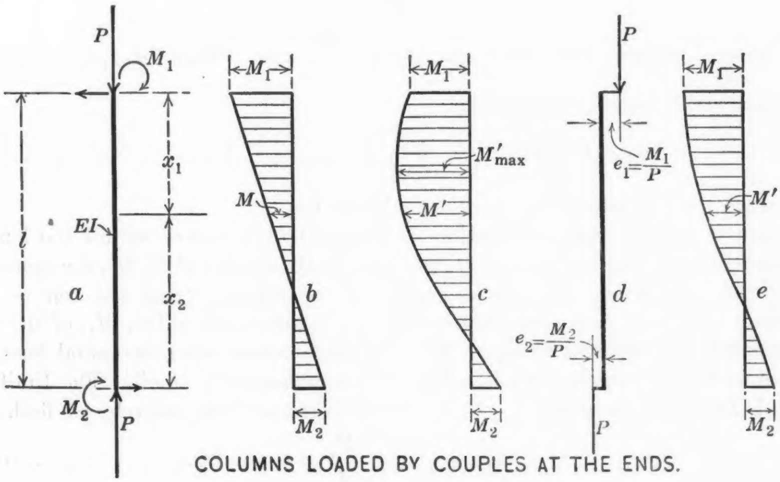
where, by the theory of Fourier series:

$$C_n = \frac{2}{l} \int_{-\frac{l}{2}}^{+\frac{l}{2}} M \cos n \left(\frac{2\,\pi\,x}{l}\right) d\,x$$

After integration, we find in this manner:

$$M_n = (-1)^{n-1} \frac{4\,M_0}{\pi^2\,n^2} \cos \frac{n\,2\,\pi\,x}{l}$$

Substitution of these values in the general Formula (12), with $x = 0$ and $x = \frac{l}{2}$, leads to Formulas (9) and (10).



COLUMNS LOADED BY COUPLES AT THE ENDS.

FIG. 4.

9.—Columns Loaded by Couples at the Ends.—The column in Fig. 4 (*a*) carries a central axial load, *P*, and at the same time is loaded by the couples, M_1 and M_2, which act at the ends, and are considered as positive in clockwise direction. The cross-section is assumed to be constant. This problem may be

treated by means of the general Formula (12) in the same manner as the preceding cases, but the particular conditions of the present problem allow a rather simple direct solution, which will now be indicated.* Let the notation for distances be as shown in Fig. 4 (*a*). The moment diagram produced by the couples alone is shown in Fig. 4 (*b*). In the combined bending and buckling action the resultant moment diagram may shape itself as shown in Fig. 4 (*c*). Evidently, the same action might be produced by the forces, P, alone, namely, when they are applied with the eccentricities indicated in Fig. 4 (*d*).

The particular object is to find the maximum values of the moments, M'. This may be done as follows: M' is expressed in terms of M_1, M_2, x_1, and P, and the deflections, y. The expression formed in this manner is substituted in the flexure equation, $E\,I\,\frac{d^2 y}{d\,x_1^{\,2}} = -\,M'$, which is then integrated, whereupon M' is found by a double differentiation. The quantities are introduced:

$$Q = \frac{\pi^2\,EI}{l^2} = \text{lowest critical axial load when ends are hinged;}$$

and,

$$k = \frac{P}{Q}$$

The formula found for M' at any point is then:

$$M' = \frac{M_1 \sin \pi \left(\frac{x_2}{l}\right) \sqrt{k} - M_2 \sin \pi \left(\frac{x_1}{l}\right) \sqrt{k}}{\sin \pi \sqrt{k}} \qquad (15)$$

M' becomes maximum or minimum when $\frac{d\,M'}{d\,x_1} = 0$. Since, $d\,x_1 = -\,d\,x_2$, the derivative becomes zero when,

$$\cos \pi \left(\frac{x_2}{l}\right) \sqrt{k} + \left(\frac{M_2}{M_1}\right) \cos \pi \left(\frac{x_1}{l}\right) \sqrt{k} = 0 \qquad (16)$$

Formula (16) is adaptable to graphical solution.

It may happen that no solution of Formula (16) occurs within the length of the column. In that case, if M_2 is numerically smaller than M_1, the moment, M', will decrease on the whole length of the column from the top to the bottom. The case is represented in Fig. 4 (*e*), where the value, M_1, of the end moment is the greatest value of M'. The case occurs when the axial load, P, is comparatively small, that is, when k is comparatively small. The limiting case is found by inserting $x_1 = 0$, $x_2 = l$ in Formula (16), whereby we find,

$$\cos \pi \sqrt{k} + \frac{M_2}{M_1} = 0 \qquad (17)$$

For each value of $\frac{M_2}{M_1}$ between $+\,1$ and $-\,1$ a corresponding limiting value of $k = \frac{P}{Q}$ may be found from Formula (17). At and below this limiting

* For the derivation of the deflected curve see A. Ostenfeld, "Teknisk Elasticitetslære", 3d ed. (1916), p. 452. The case is important with regard to the question of shearing stresses in columns (*ibid*, p. 453).

value, we have max. $M' = M_1$. Above the limiting value of k, the values of x_1 and x_2 found from Formula (16), are to be substituted in Formula (15), thereby giving a value of max. M'.

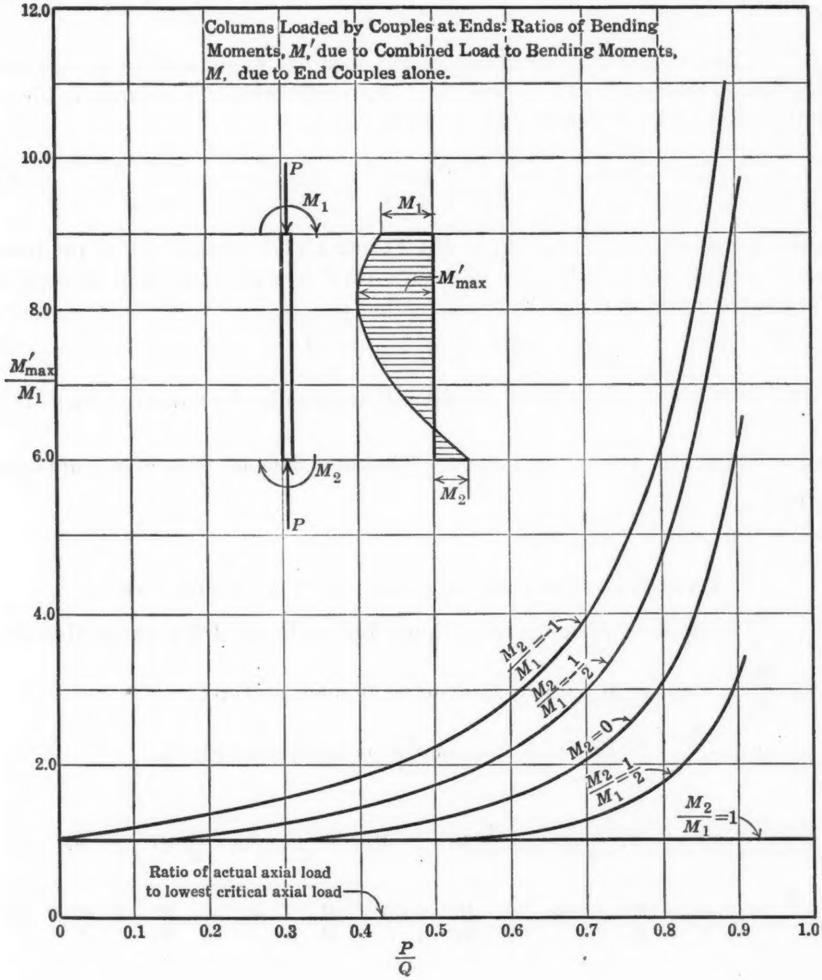


FIG. 5.

Three special cases will be examined, namely, $M_2 = -M_1$, $M_2 = 0$, and $M_2 = M_1$ respectively. First, assume $M_2 = -M_1$: In this case the moment due to the end couples alone is constant throughout the length. The case may be produced by eccentric loading with the same eccentricity, to the same side, at the top, and at the bottom. Formula (16) gives, in this case, $x_1 = x_2 = \frac{l}{2}$,

that is, as would be expected, the greatest moment occurs at the center. Formula (15) gives then:

$$\max.\ M'_{M_2 = -M_1} = \frac{2\ M_1 \sin \frac{\pi}{2} \sqrt{k}}{\sin \pi \sqrt{k}} = M_1 \sec \frac{\pi}{2} \sqrt{k} \ \ldots\ldots (18)$$

This equation is a form of the secant formula which applies to eccentric loading. The second special case, $M_2 = 0$, occurs when the column is hinged at the lower end. Formulas (15) and (16) give:

$$\max.\ M'_{M_2 = 0} = \frac{M_1}{\sin \pi \sqrt{k}} \text{ at point } x_2 = \frac{l}{2 \sqrt{k}} \ldots\ldots (19)$$

In the third special case, $M_1 = M_2$, Formula (17) gives $k = 1$ as the limiting value. It follows that for all values of P less than the critical load, Q, the greatest moment occurs at the ends, that is,

$$\max.\ M'_{M_2 = M_1} = M_1 \ \ldots\ldots\ldots\ldots\ldots\ldots\ (20)$$

Table 3 gives solutions of the preceding formulas for various values of $\frac{M_2}{M_1}$ and for values of $\frac{P}{Q}$ between 0 and 1. The same data are presented graphically in Fig. 5.

TABLE 3.—COLUMNS BENT BY COUPLES AT THE ENDS.

$\frac{x_1}{l}$ = Ratio of Distance from Upper End to Point of Maximum Moment;

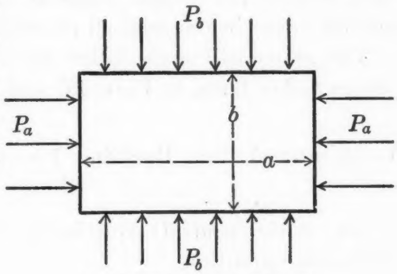
$\frac{M'}{M_1}$ = Ratio of Maximum Moment to Moment at Top;

$Q = \frac{\pi^2 EI}{l^2}$; otherwise, the notation is as given in Fig. 5.

$\frac{P}{Q}$	$M_2 = -M_1$		$M_2 = -\frac{1}{2} M_1$		$M_2 = 0$		$M_2 = \frac{1}{2} M_1$		$M_2 = M_1$	
	$\frac{x_1}{l}$	$\frac{M'}{M_1}$	$\frac{x_1}{l}$	$\frac{M'}{M_1}$	$\frac{x_1}{l}$	$\frac{M'}{M_1}$	$\frac{x_1}{l}$	$\frac{M'}{M_1}$	$\frac{x_1}{l}$	$\frac{M'}{M_1}$
.0	....	1.00	0	1	0	1	0	1	0	1
0.1	0.5	1.14	0	1	0	1	0	1	0	1
0.111	....	....	0	1.00	......	......	....	....	......	......
0.2	0.5	1.31	0.23	1.06	0	1	0	1	0	1
0.25	....	....	....	....	0	1.00	....	....	......	......
0.3	0.5	1.54	0.34	1.20	0.09	1.01	0	1	0	1
0 4	0.5	1.83	0.39	1.41	0.21	1.09	0	1	0	1
0.444	....	....	....	....	......	......	0	1.00	......	......
0.5	0.5	2.25	0.43	1.72	0.29	1.26	0.06	1.01	0	1
0.6	0.5	2.89	0.45	2 18	0.35	1.54	0.16	1.08	0	1
0.7	0.5	3.94	0.47	2.96	0.40	2.04	0.25	1.25	0	1
0.8	0 5	6.06	0.48	4.55	0.44	3.07	0.33	1.69	0	1
0.9	0.5	12.5	0.49	9.35	0.47	6.25	0.42	3.20	0	1
1.0	0.5	∞	0.50	∞	0.50	∞	0.50	∞	......	∞
										∞

III.—Slabs and Similar Structures.

10.—Formulas Applying to Rectangular Homogeneous Slabs.—A rectangular slab may buckle under the influence of loads at the edges acting in the original plane of the slab. Two examples of this kind were mentioned in the Introduction (Fig. 1, (K) and (L)). In the one case, the load was perpendicular to the edge, in the other parallel to the edge. Until the deflections have become quite large, the action is a typical astatic elastic action (pure buckling). Loads from the side, perpendicular to the slab, acting alone, produce orthostatic action, that is, ordinary flexure with the deformations proportional to the loads. A combination of the end loads in the plane of the slab and the side loads produces mixed or heterostatic action. When the frames of a ship are rather widely spaced, such actions may become important in the shell (deck, sides, and bottom). Cases of buckling of rectangular slabs have been analyzed by Messrs. G. H. Bryan, S. Timoshenko, and others, and reference is made to the treatments by these authors (Bibliography, Part VII, Sections A and J). The material presented by the writer will include some cases of buckling or astatic action as well as some cases of mixed or heterostatic action.



SLAB WITH BUCKLING LOADS.
FIG. 6.

Let (Fig. 6), a = longer span;

b = shorter span;

$\alpha = \dfrac{b}{a}$

EI = modulus of elasticity times moment of inertia of cross-section per unit width;

K = Poisson's ratio of lateral contraction to longitudinal elongation when a longitudinal load acts alone;

w = uniformly distributed load per unit area, perpendicular to the surface of the slab;

P_a = uniformly distributed compressive end load per unit width, acting at the short edges in the direction of the long span. A negative P_a represents tension;

P_b = uniformly distributed compressive end load per unit width, acting at the long edges in direction of the short span;

Q_a and Q_b = critical values of the end loads found by increasing the original values of P_a and P_b by the same ratio until buckling can take place without the aid of the surface load, w;

M = moment in slab per unit width in the orthostatic action, in which the surface load, w, is the only external load;

M' = moment in the mixed or heterostatic action in which end loads, such as P_a and P_b, act at the same time as the surface load, w;

$M'' = M' - M$ = increase of moment produced by application of the end load when the surface load is already acting;

M_b, M'_b, M''_b = values of M, M', and M'' at the center of the slab, for bending in the short span, b (that is, in an element of section perpendicular to the short span, b);

m, n = positive integers.

The writer will confine himself to cases in which the only end loads in the plane of the slab are those perpendicular to the edges, such as, P_a and P_b in Fig. 6. It is assumed that each rectangular panel can be treated separately, and, accordingly, only one single independent panel will be considered. The slab is assumed to be homogeneous, isotropic, and sufficiently thin so that it may begin to buckle without exceeding the elastic limit. It is also assumed that the deflections have not progressed so far as to affect the uniformity of the distribution of the pressures in the plane of the slab. The slab under consideration has simple supports along the four edges, and these supports hold the edges in the original plane of the slab.

The writer will again follow the plan of presenting results first. The derivations follow later, in Parts IV and V.

A.—Astatic Action, Buckling Due to End Loads Alone, When the Slab is Simply Supported.

An astatic (neutral) equilibrium may occur when the end loads, Q_a and Q_b, satisfy the condition:

$$\frac{m^2 Q_a}{a^2} + \frac{n^2 Q_b}{b^2} = \frac{\pi^2 E I}{1 - K^2}\left[\frac{m^2}{a^2} + \frac{n^2}{b^2}\right]^2 \qquad (21)$$

where $m, n = 1, 2, 3 \ldots\ldots$*. The lowest critical values of Q_a and Q_b have the greatest interest, as they are the values at which the actual buckling would take place if the load was increased gradually from zero. In buckling caused by the lowest critical loads, one of the integers, m and n, is always equal to 1. The values of m and n, and the corresponding solutions of Formula (21), will now be indicated in some definite cases.

Case 1.—Square slab, $a = b$:

a.—Q_b positive, $Q_a = 0$ (compression in direction of one span only):

$$m = n = 1$$

Formula (21) gives

$$Q_b = \frac{4\,\pi^2 E I}{(1 - K^2) b^2} \qquad (22)$$

b.—Q_a and Q_b both positive (both compressive loads):

$$m = n = 1$$

In this case, Formula (21) becomes,

$$Q_a + Q_b = \frac{4\,\pi^2 E I}{(1 - K^2) b^2} \qquad (23)$$

---

* Formula (21) was given by G. H. Bryan, *Proceedings*, London Math. Soc., v. 22 (1890), p. 57; see, also, A. E. H. Love, "Mathematical Theory of Elasticity" (1906), p. 529.

c.—Q_b positive, $Q_a = -Q_b$ (that is, Q_a is a tension equal in magnitude to the compression, Q_b). This case is of some importance because it represents the condition of pure shear in the directions of the diagonals:

$$m = 1, n = 2$$

Substitution of these values in Formula (21) gives:

$$Q_b = \frac{25\ \pi^2\ E\ I}{3\ (1 - K^2)\ b^2} \quad \text{(24)}$$

d.—Q_b positive, $Q_a = -2Q_a$:

$$m = 1, n = 2;\ Q_b = \frac{25\ \pi^2\ E\ I}{2\ (1 - K^2)\ b^2}$$

e.—General case, Q_b positive, ratio $\frac{Q_a}{Q_b}$ any value:

$$m = 1;\ Q_b = \frac{\pi^2\ E\ I}{(1 - K^2)\ b^2} \frac{(n^2 + 1)^2}{n^2 - \left(-\frac{Q_a}{Q_b}\right)} \quad \text{(25)}$$

$$n = 1 \text{ when } -\frac{Q_a}{Q_b} < 0.43$$

$$n = 2 \text{ when } 0.43 < -\frac{Q_a}{Q_b} < 2.33$$

$$n = 3 \text{ when } 2.33 < -\frac{Q_a}{Q_b} < 5.30$$

Case 2.—Rectangular slab, $a > b$. $Q_a = 0$, that is, the load is in the direction of the short span only:

$$m = n = 1;\ Q_b = \frac{\pi^2\ E\ I\ (1 + \alpha^2)^2}{(1 - K^2)\ b^2} \quad \text{(26)}$$

It is noted that the limiting case, $\alpha = 1$, leads to Formula (22). $\alpha = 0$, that is $a = \infty$, and Poisson's ratio, $K = 0$, gives the usual Euler formula for columns, $Q = \frac{\pi^2\ E\ I}{b^2}$.

Case 3.—Rectangular slab, $a > b$. $Q_b = 0$, that is, the load is in the direction of the long span only:

$$n = 1;\ Q_a = \frac{\pi^2\ E\ I}{(1 - K^2)\ b^2} \frac{(m^2\ \alpha^2 + 1)^2}{m^2\ \alpha^2} \quad \text{(27)}$$

When $\frac{1}{\alpha}$ is an integer, then $m = \frac{1}{\alpha}$. Otherwise m = either the nearest integer above or the nearest integer below $\frac{1}{\alpha}$. This result may be proved by considering m as varying continuously; then Q_a in Formula (27) is found to have a minimum when $m = \frac{1}{\alpha}$. This result means physically that in the buckling of the center line of the long span, the half-wave length (distance between points of inflection) has a tendency to become equal to the shorter

span of the slab. Under any circumstance, $m\ \alpha = 1$ substituted in Formula (27) defines a lower limit of the critical load, namely,

$$Q_a = \frac{4\ \pi^2\ E\ I}{(1 - K^2)\ b^2} \qquad \text{(27a)}$$

In most cases Formula (27*a*) would replace Formula (27) with good approximation.

Case 4.—General case: Rectangular slab, $b < a$, $\frac{Q_a}{Q_b}$ = any value. The general Formula (21) is to be used. It was stated that in order to obtain the lowest critical loads one of the integers, m and n, must always be equal to 1. The following rule applies:

$$m = 1 \text{ when } \frac{Q_a}{Q_b} < 0.5,\ Q_b > 0;$$
$$n = 1 \text{ when } Q_a \geqq 0.$$

The general Formula (21) is the equation of a straight line in the co-ordinate system, Q_a, Q_b (Fig. 7). By keeping m constant, but varying n, a set of straight lines is obtained all of which are tangents to the same parabola with the axis parallel to the Q_a-axis. In the same way, variation of m while n is kept constant, gives a set of tangents to a parabola with the axis parallel to the Q_b-axis. Two such sets of tangents, one for $m = 1$ and one for $n = 1$, form a polygon which defines the lowest critical values, Q_a and Q_b, for all ratios, $\frac{Q_a}{Q_b}$. The two parabolas pass through the origin with their tangents making angles of 45° with the axes. The parabolas are open backward and downward. Fig. 7 shows the parabolas for the cases of the square slab, $a = b$, and the rectangular slab, $a = 2\ b$. The vertices of the polygons represent conditions under which two different buckling waves may be superimposed, one on the other, so that a double, neutral equilibrium occurs at those particular loads.

B.—Orthostatic Flexure, Bending Without End Loads in the Plane of the Slab, the Slab Being Simply Supported Along the Four Edges.

The surface load, w, is acting, but there are no pressures from the ends. It will be assumed temporarily that Poisson's ratio, K, of lateral contraction to longitudinal stretch is zero. The moment at the center in Span b is then found to be given approximately by the following formula:

$$M_b = \frac{\frac{1}{8}\ w\ b^2}{1 + \alpha^2 + 1.4\ \alpha^4} \qquad (28)$$

In a slab which is square or nearly square greater moments occur along and across the diagonals at the corners than at the center. Formula (29) may be used with advantage in many cases. It gives a reasonable approximation to the maximum moment in the slab. In slabs which are nearly square, it gives somewhat too low stresses, but a slight local yielding at the corners

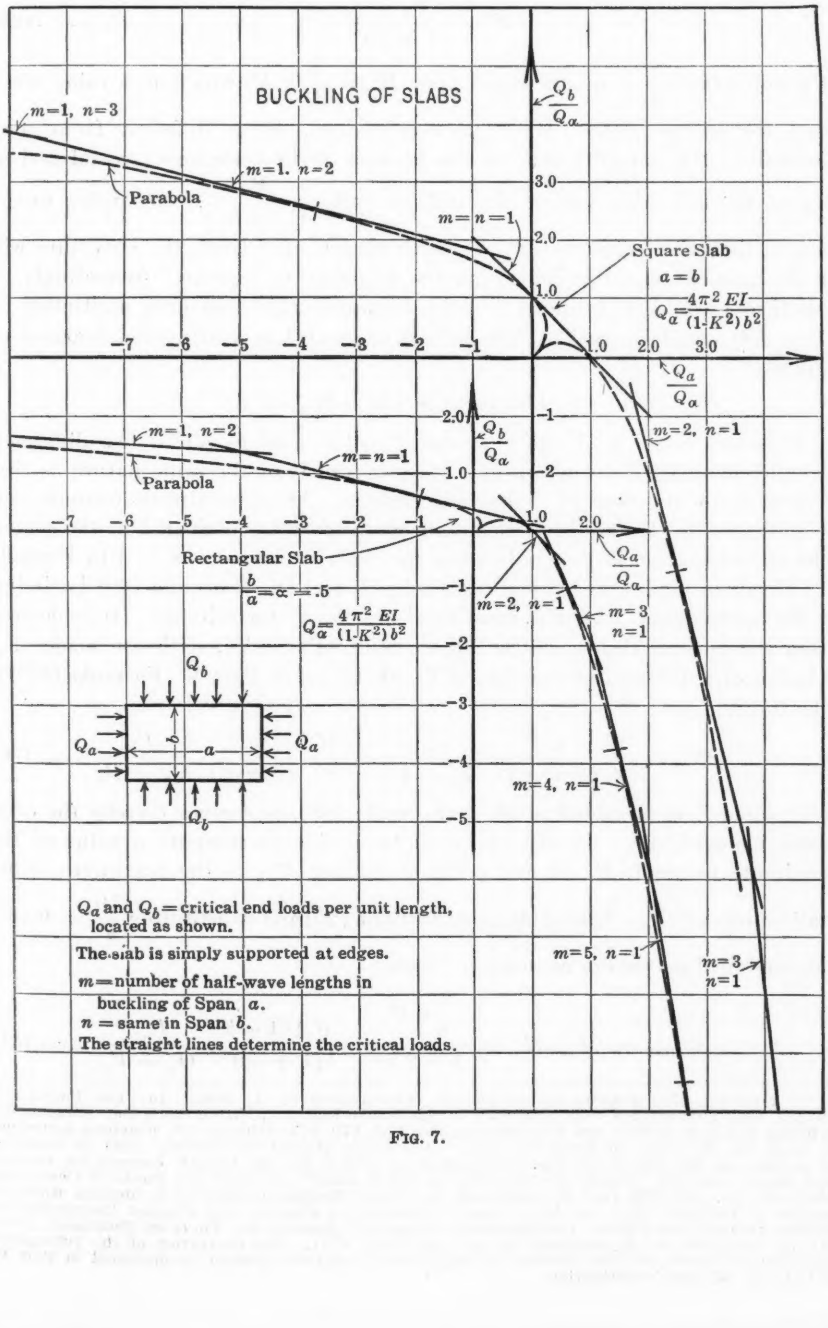

BUCKLING OF SLABS
m=1, n=3
m=1. n=2
Parabola
m=n=1
Square Slab
a=b
$Q_a = \frac{4\pi^2 EI}{(1-K^2)b^2}$
$\frac{Q_b}{Q_a}$
$\frac{Q_a}{Q_a}$
m=2, n=1
m=1, n=2
m=n=1
Parabola
Rectangular Slab
$\frac{b}{a} = \alpha = .5$
$Q_a = \frac{4\pi^2 EI}{(1-K^2)b^2}$
m=2, n=1
m=3 n=1
m=4, n=1
m=5, n=1
m=3 n=1
Q_b
Q_a
a
b
Q_a and Q_b = critical end loads per unit length, located as shown.
The slab is simply supported at edges.
m = number of half-wave lengths in buckling of Span a.
n = same in Span b.
The straight lines determine the critical loads.

FIG. 7.

will re-distribute the stresses until values almost similar to those given by the formula are obtained:*

$$M = \frac{\frac{1}{8} w b^2}{1 + 2\alpha^3} \qquad \text{.........................(29)}$$

It is noted that $\frac{b}{a} = \alpha = 1$ would give $M = \frac{1}{24} w b^2$, which is a value often used for square slabs. When Poisson's ratio, K, is different from zero, Formulas (28) and (29) indicate the product of the curvature (second derivative of the deflections) times the stiffness ratio, $\frac{E I}{(1 - K^2)}$. According to the "strain theory" of rupture—St. Venant's theory—$E I$ times the curvature will be the moment quantity indicating the tendency to rupture. Accordingly, if this theory could be assumed to hold, Formulas (28) and (29) multiplied by $(1 - K^2)$ could be used as "theoretical moments" measuring the nearness of rupture.

C.—Heterostatic (Mixed) Action.

Poisson's ratio, K, is again assumed to be equal to zero. The difference brought about by Poisson's ratio not being zero is of the same nature as that described for the case of orthostatic flexure. An approximate formula will be indicated for the additional bending moment, M''_b, in Span b at the center. The approximation is close only when the combination $m = n = 1$ in Formula (21) leads to the lowest critical end loads, Q_a and Q_b. Formula (30) is similar to the approximate formulas mentioned previously for columns. It is derived from the general theory which follows later (Part IV), with use made of a solution of a differential equation of flexure given in Part V (Formula (150)). The formula is:

$$M''_b = \frac{16}{\pi^4} \frac{w b^2}{(1 + \alpha^2)^2} \frac{P_a}{Q_a - P_a} = \frac{16}{\pi^4} \frac{w b^2}{(1 + \alpha^2)^2} \frac{P_b}{Q_b - P_b} \quad \text{....} \; (30)$$

This value is to be added to M_b in Formula (28), and gives thereby the combined moment, M'_b. It will be on the safe side to compute a value of the maximum moment, M', at any point by adding M''_b to the maximum orthostatic moment, M. Taking M from Formula (29) and substituting $\frac{16}{\pi^4} = 0.164$, the combined maximum moment one finds:

$$M'_{\text{max.}} = M + M'' = \frac{\frac{1}{8} w b^2}{1 + 2\alpha^3} + \frac{0.164\, w b^2}{(1 + \alpha^2)^2} \frac{P_a}{Q_a - P_a} \quad \text{.....} \; (31)$$

* Formula (28) follows rather closely values given by A. Nádai, in "Die Formänderungen und Spannungen von rechteckigen elastischen Platten", *Mitteilungen* über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens, Vol. 170–171, table, p. 34, provided correction is made for the effect of Poisson's ratio. A formula similar to Formula (28) is indicated in a note on the design of reinforced concrete, issued by the French Council on Bridges and Roads, entitled, "Calcul des Hourdis en Béton Armé", *Annales des Ponts et Chaussées*, 1912, IV, pp. 469–529 (see in particular, p. 474). Mesnager indicates a formula which is similar to Formula (29), see his memoir, "Moments et Flèches des Plaques Rectangulaires Minces Portant une Charge Uniformément Répartie", *Annales des Ponts et Chaussées*, 1916, III, pp. 313–438 (see in particular p. 436 and Plate VII). The derivation of the differential equation of flexure, on the solutions of which these analyses depend, is indicated in Part V, Article 31, of this investigation.

Formula (31) applies with the same limitations as Formula (30). In the case of a square slab, we have $\alpha = 1$. In that case, Formula (31) gives:

$$M'_{\max.} = \frac{1}{24}\, w\, b^2 + 0.041\, w\, b^2 \frac{P_a}{Q_a - P_a}$$

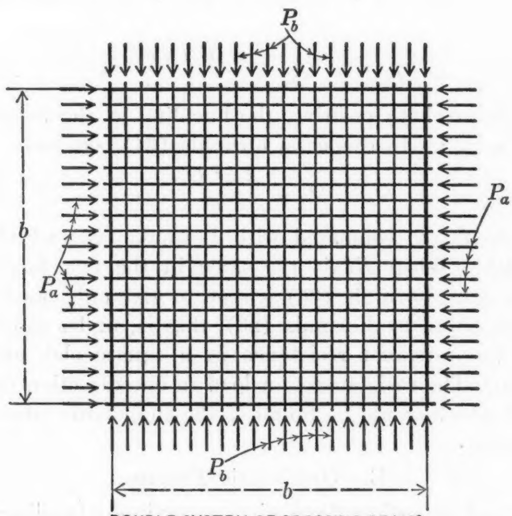
that is, with sufficiently close approximation,

$$M'_{\max.} = \frac{w\, b^2}{24} \frac{Q_a}{Q_a - P_a} \quad \ldots\ldots\ldots\ldots\ldots\ldots (32)$$

When one of the integers m and n is different from 1, Formulas (30), (31), and (32), with Q_a and Q_b taken as the lowest critical loads, will lead to values which are in most cases decidedly higher than the actual values. However, in all such cases, as well as in those in which Formulas (30) to (32) apply, Formula (33) will be on the safe side, or only slightly on the unsafe side:

$$M' = M \frac{Q_a}{Q_a - P_a} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

Formula (33) has the same form as the current approximate Formula (2b) in Article 3 and Formula (4a) in Article 4.



DOUBLE SYSTEM OF CROSSING BEAMS WITH BUCKLING LOADS.

FIG. 8.

11.—Formulas Applying to a Double System of Crossing Beams Connected at Cross-Points.—Fig. 8 shows two systems of beams crossing one another at right angles and arranged in a square. The beams are simply supported at the edges of the square so that these edges must remain in their original plane. The beams are connected at the cross-points so as to make the deflections of the two beam systems the same without causing any torsional moments in the beams. A great number of beams are assumed, uniformly spaced and with uniform stiffness. The end loads on each beam system are evenly distributed, and are transmitted directly to each beam as axial loads, so as to give a

constant total axial pressure in any section of any beam. The structure resembles a slab, but the difference lies in the lack of torsional resistance in the double-beam system. This quality is demonstrated by the fact that one could lift one of the corners out of the original plane, and thereby produce a double curved surface, without putting stresses in any of the beams. It may be interesting to investigate this case for the purpose of comparison with the homogeneous slab. A square concrete slab with reinforcing bars parallel to the edges has properties intermediate between those of the homogeneous slab and those of the double-beam system, and the resistance of two-way reinforced concrete against buckling might be estimated by comparison with the two extreme cases.

The same notation will be used as in Article 10. The loads, moments, and the stiffness factor, $E\,I$, are indicated per unit width of section. It will be assumed that the spacing of the beams is so close that the area elements of the structure may be considered as continuous.

A.—Astatic Action.

The general formula for buckling, to be compared with Formula (21) in Article 10, is

$$m^2\,Q_a + n^2\,Q_b = \frac{\pi^2\,E\,I\,(m^4 + n^4)}{b^2} \quad\text{..............}(34)$$

When Q_a and Q_b are both positive, the buckling at the lowest load is represented by $m = n = 1$. In this case, Formula (34) becomes:

$$Q_a + Q_b = \frac{2\,\pi^2\,E\,I}{b^2} \quad\text{......................}(35)$$

Formula (35) should be compared with Formula (23) which applies to the homogeneous slab. When EI is the same in the two formulas, and K in Formula (23) is zero, Formula (35) is seen to give only one-half the value of the critical loads found by Formula (23), that is, under those circumstances, elimination of the torsional resistance in a square slab cuts the buckling resistance in two. The resistance to a load in one direction only is still twice the critical load which would be found if the supporting edges parallel to the load were removed.

B.—Orthostatic Flexure.

The surface load, w, acting alone, gives a maximum bending moment, occurring at the center, equal to

$$M_{\text{max.}} = 0.077\,w\,b^2 \quad\text{....................}(36)$$

(to be compared with $0.037\,wb^2$ in the square homogeneous slab, as given by Formula (28) with $\alpha = 1$). The method of deriving Formula (36) will be indicated in Part V.

C.—Heterostatic Action.

When $m = n = 1$ gives the smallest values of Q_a and Q_b in Formula (34), Formula (37) may be used for the moment at the center per unit width:

$$M'_{\text{max.}} = 0.077\,w\,b^2\left(1 + 1.07\,\frac{P_a}{Q_a - P_a}\right) \quad\text{.........}(37)$$

(to be compared with Formula (32) applying to the homogeneous slab).

IV.—General Theory of Structural Actions in Which the Stresses Are Not Proportional to the Loads, Although Hooke's Law Applies in Each Element of the Structure, and Although the Deformations Are Small.

12.—The Energy Equation, the Loads, and the Parameters.—In the Introduction, Article 1, definitions were given of three general types of elastic action: Orthostatic or regular action, in which the stresses are proportional to the loads; astatic action in which pure buckling occurs; and heterostatic or mixed action. In the two latter types of action, the stresses are not, in general, proportional to the load. The two assumptions common to all cases considered are recalled, namely, that Hooke's law of proportionality of stresses and strains applies in each structural element, and that the deformations are small. It is now proposed to investigate some of the general features of astatic and heterostatic actions. The energy principle, or the principle of least action, is found to be particularly effective for this purpose. This principle makes it possible to express the conditions of equilibrium by one condition or equation—the energy equation. The following notation will be used:

Let U = total internal elastic energy stored in the structure by the deformation, or, the elastic potential energy of the structure. This quantity is called by many writers the resilience* of the structure, and this term will be used in this paper.

P = external load in the generalized sense, or "generalized force". P is a quantity which may represent one force or a group of forces acting on the structure. When all the forces in this group are increased by a certain proportion, this increase is represented by an increase of P by the same proportion. P, therefore, is a quantity measuring the intensity of load in the whole group of forces. The load, P, is the "generalized force" which was introduced by Lagrange in his general equations of dynamics.

δ = displacement in the direction of the load, P. If P is a single force, δ is the component of the linear displacement of the point where P is applied, in the direction of P. If P represents a group of forces, δ is such a quantity that $P\delta$ is the work done by the load, P, while the elastic displacements increase from zero to their final values, P being constant during this process. For example, if P is a torque (couple), δ is the angular rotation (in radians) of the structural part on which the couple is acting. If the load, P, consists of two forces each equal to P, acting against one another at the end points of a certain

* In Johnson's "Materials of Construction", 1919 edition, p. 38, resilience is defined as "the work which a body can do in springing back after a deforming force has been removed". The present use of the term is in accordance with this definition. See, also, A. E. H. Love, "Mathematical Theory of Elasticity", 1906 edition, p. 170, and J. A. Ewing. "Strength of Materials", 1906 edition, p. 15. There has been some disagreement as to the use of the term, resilience; for example, E. S. Andrews ("Strength of Materials", 1915, pp. 33, 272) uses the term in the sense of energy per unit volume.

distance, δ is the shortening of this distance. If the load, P, consists of two forces each equal to $\frac{1}{2}$ P, acting against one another at the end points of a certain distance, δ is one-half the shortening of the distance, because only then could $P\delta$ be equal to the work of the load. If the load, P, consists of the individual forces, $P_1, P_2, \ldots\ldots P_n \ldots\ldots$, in the direction of which the displacements are, $\delta_1, \delta_2, \ldots\ldots \delta_n \ldots\ldots$, δ is found from the equation $P\delta = \Sigma P_n \delta_n$. In Lagrange's dynamics, δ is the generalized displacement, or, the change of a generalized co-ordinate.

$L =$ the difference between the internal energy or resilience of the structure and the work which is done by the external forces during the deformation, if the external forces remain constant during this process. That is, when P is the only load, $L = U - P\delta$. L may then be considered as the total potential energy of the structure and loads.

$u_1, u_2 \ldots\ldots u_m \ldots\ldots u_n \ldots\ldots =$ parameters of the structure which define the elastic displacements, or, in terms of Lagrange's dynamics, "generalized co-ordinates" of the structure. Unless otherwise stated, zero displacements correspond to $u_1 = u_2 \ldots\ldots = u_m \ldots\ldots u_n \ldots\ldots = 0$. Definite values assigned to each of the parameters will define completely the elastic changes of shape, that is, the displacements are functions of the parameters, u. In most cases, the number of parameters is infinite. In the case of an originally straight, incompressible, but flexible column, the constants in the Fourier series, expressing the deflections in terms of the distance along the column, may be used as parameters (compare Article 8) and they constitute a complete set. U and δ are functions of the parameters.

The general condition of equilibrium when P is the only load, may now be written:

$$L = U - P\delta = \pm \text{ min.} \quad \ldots\ldots\ldots\ldots (38)$$

The $\pm$ min. stands for "minimum or maximum or condition of zero derivatives". A minimum of L corresponds to a stable equilibrium, a maximum to an unstable equilibrium. Since the cases of stable equilibrium, giving a minimum, are the more importan. –in orthostatic action this case will be found to be the only one possible—and since the definite conditions derived by differentiation of the equation are the same whether a maximum or a minimum occurs, the condition of equilibrium in the standard form may be written:

$$L = U - P\delta = \text{min.} \quad \ldots\ldots\ldots\ldots (39)$$

where the "minimum" on the right side is understood as a $\pm$ min., namely, usually a minimum, but, in special cases, a maximum. When the minimum occurs, Formula (39) expresses the "principle of least action". By differ-

entiating Formula (38) or Formula (39), with respect to the parameters, it will be found that:

$$\ldots\frac{\partial L}{\partial u_m}=0\ldots\ldots\frac{\partial L}{\partial u_n}=0\ldots\ldots\ldots\ldots(40)$$

giving one equation for each parameter. The set of equations, Formula (40), is a complete equivalent of the energy equation, Formula (38) or Formula (39).

13.—The Internal Actions or Generalized Stresses, and the Elastic Potential Energy or Resilience.—The stresses at any one point measure the intensity of the internal elastic action. The stresses, however, are not the only quantities which can be used for this purpose; for example, in a beam which is bent by transverse forces only, the bending moment, M, is a measure of the intensity of action in a longitudinal element of length, ds, enclosed between two consecutive cross-sections. When EI is the product of the modulus of elasticity and the moment of inertia of the cross-section, $\frac{M^2\,ds}{2\,E\,I}$ is the internal energy or resilience of the element. If, in addition, there is an axial force, P, this force gives an additional stress uniformly distributed over the area, A, of the cross-section, and creates in the same structural element an additional element of energy equal to $\frac{P^2\,ds}{2\,E\,A}$. Therefore, P is also a "quantity measuring internal action". Usually, in beams and columns, the influence of the shearing stresses on the deformations is negligible. That is, the bending moments, M, together with the axial load, P, determine completely the internal energy or resilience of the column. M and P therefore may be called stresses in the generalized sense. In the general case, the action of a structure may be described in terms of some set of "internal actions" or "generalized stresses", R, which have such properties that the element of energy is written $\frac{1}{2}\,r\,R^2\,ds$, in which r is a function of the location and s measures "extension of the structure". The total elastic potential energy or the resilience is then expressed:

$$U=\frac{1}{2}\int r\,R^2\,ds\ldots\ldots\ldots\ldots(41)$$

where the integration is to be extended over the whole structure, and is to include all the elements of energy. It is always possible to indicate quantities such as R, which give an energy expression in the Formula (41).* As in the

*Let $d\,V$ = volume element of a homogeneous isotropic elastic solid;
X, Y, Z = normal stresses in the directions, x, y, z;
S_{yz}, S_{zx}, S_{xy} = the corresponding shearing stresses;
E = modulus of elasticity;
K = Poisson's ratio of lateral contraction to longitudinal elongation.

Then, the elastic potential energy of the solid may be expressed:

$$U=\frac{1}{2}\int\Big[\frac{1}{E}(X^2+Y^2+Z^2)-\frac{2\,K}{E}(Y\,Z+Z\,X+X\,Y)+\frac{2\,(1+K)}{E}(S^2_{yz}+S^2_{zx}+S^2_{xy})\Big]\,d\,V\ldots\ldots(41a)$$

case of the axially compressed beam, the same element, *ds,* may be included more than once in the integral of the resilience.

It will be assumed that the actions, *R,* are linear functions, and unless otherwise stated, linear homogeneous functions of the parameters, $u_1, u_2 \ldots\ldots$ When only small deformations of the structure are considered, it is always possible to choose a set of parameters satisfying this condition; and when one set of such parameters has been found, this set may be replaced by any other set in which each parameter is a linear homogeneous function of the parameters of the first set. Formula (42) may then be written:

$$R = R_1 u_1 + R_2 u_2 + \ldots\ldots + R_m u_m \ldots\ldots + R_n u_n + \ldots\ldots \quad (42)$$

where $R_1 \ldots\ldots R_n \ldots$, are functions of the location, that is, functions of *s*. Substituting Formula (42) in Formula (41) the expression for the internal energy or resilience is found:

$$U = \left(\frac{1}{2}\right) \underset{m}{\Sigma} u^2_m \int r R^2_m \, ds + \underset{mn}{\Sigma\Sigma} u_m u_n \int r R_m R_n \, ds \ldots\ldots \quad (43)$$

where $m = 1, 2 \ldots\ldots, n = 1, 2, \ldots\ldots$, and $m \neq n$, and where the integrations are extended so as to include all elements of energy. The integrals in the summations may be considered as "constants of the structure", and will be denoted as follows:

$$U_m = \int r R^2_m \, ds; \; U_{mn} = \int r R_m R_n \, ds \ldots\ldots\ldots\ldots \quad (44)$$

The resilience is then expressed:

$$U = \frac{1}{2} \Sigma U_m u^2_m + \Sigma \Sigma U_{mn} u_m u_n \ldots\ldots\ldots\ldots \quad (45)$$

where $m \neq n$. It is noted that *U* is a quadric in the parameters (a quadric is defined as a homogeneous function of second degree).

The additional notation introduced is summarized here as follows:

Let R = generalized stress, or, quantity measuring internal action;

ds = dimension of the structural element;

r = function of the location, depending on the stiffness of the element in such a way that $\frac{1}{2} r R^2 ds$ is equal to the corresponding element of energy or resilience;

As Formula (41*a*) contains the products, *YZ, ZX,* and *XY,* the stresses *X, Y,* and *Z* cannot be used as *R*'s in Formula (41), except when two of these stresses are zero. Formula (41*a*), however, may be written in the form, Formula (41), for example, as follows:

$$U = \frac{1}{2} \int \left[\frac{1 - 2K}{3E} (X + Y + Z)^2 + \frac{1 + K}{2E} (X - Z)^2 + \frac{1 + K}{6E} (X - 2Y + Z)^2 + \frac{2(1 + K)}{E} (S^2_{yz} + S^2_{zx} + S^2_{xy}) \right] dV = \frac{1}{2} \int (\Sigma r R^2) \, ds \ldots\ldots \quad (41b)$$

That is, Formula (41) may be used with the following set of generalized stresses, *R*, corresponding to the volume element, *dV* or *ds:*

$$R^{I} = X + Y + Z; \; R^{II} = X - Z; \; R^{III} = X - 2Y + Z; \; R^{IV} = S_{yz}; \; R^{V} = S_{zx}; \; R^{VI} = S_{xy} \ldots\ldots \quad (41c)$$

$R_1, R_2 \ldots\ldots R_m \ldots\ldots =$ coefficients of the parameters in the linear expression of R. These coefficients are functions of the location;

$\ldots\ldots U_m \ldots\ldots U_{mn} \ldots =$ constants of the structure, which are coefficients in the quadric expressing the resilience.

14.—The Case of Orthostatic Action.—The energy equation, Formula (39), and its equivalent, Formula (40), give equations of the form:

$$\frac{\partial L}{\partial u_m} = \frac{\partial U}{\partial u_m} - P \frac{\partial \delta}{\partial u_m} = 0 \ldots\ldots\ldots\ldots\ldots\ldots (46)$$

namely, one such equation for each parameter. Since U is a quadric in the parameters (Formula (45)), the derivatives $\frac{\partial U}{\partial u_m}$ are linear homogeneous functions of the parameters. In orthostatic action, the stresses, and, therefore, the parameters, are linear functions of the load, hence, if P is to give orthostatic action, the derivatives, $\frac{\partial \delta}{\partial u_m}$, must be independent of the u's; that is, they must be constants; or, δ in orthostatic action is a linear function of the parameters. It is assumed, in accordance with the general rule which was adopted, that the state of zero stresses and zero deformations is represented by zero values of the parameters. Then δ is a linear homogeneous function of the parameters.

*15.—Astatic Action, Astatic Parameters, and the Theorem of Orthogonality.**—It is now assumed that P is a load which can produce astatic action. Let Q_m be a value of P at which buckling is possible; that is, Q_m is a "critical value" of P. When P becomes equal to Q_m, then the structure enters into a state of astatic elastic equilibrium, that is, an equilibrium maintained at constant load throughout a continuous range of configurations of the structure. The parameter, u_m, will be chosen in a manner which correlates it peculiarly with the buckling under the load, Q_m. Three configurations or shapes of the structure will be considered, which will be called Configurations No. 1, No. 2, and No. 3. Configurations No. 1 and No. 2 are selected among the shapes of the structure which occur in the process of buckling under the load, Q_m. The change of the structure from Shape No. 1 to Shape No. 2 will be represented by a certain set of changes in the stresses or generalized stresses. The change of each generalized stress, R, will be expressed as the product, $R_m u_m$ of a fixed chosen value, u_m, times a factor, R_m. One such factor, R_m, belongs to each stress which is represented. The whole set of changes of stresses is defined completely when the set of such factors, R_m, and the quantity, u_m, are given. Now, assume that the value of u_m is changed, and that each factor, R_m, is multiplied by the new value of u_m. The products define a set of changes of the stresses. When the changes are measured from the initial configuration, No. 1, the set of changes defines a possible shape of the structure. This shape will be recorded as Configuration No. 3.

* It might seem that a simpler definition of the "astatic parameter" than the one given here could be found. The apparent complication is due to the fact that the definition is to apply, not only to the simplest cases, but also to the cases described later as "double astatic equilibrium" (Article 16) and "mixed astatic action" (Article 23).

The changes of stresses (generalized stresses or internal actions) caused by the transformation from Shape No. 1 to Shape No. 3 are proportional to the corresponding changes which occur when the structure is transformed from Shape No. 1 to Shape No. 2. The ratio between the two sets of changes is the ratio of the corresponding values of u_m. The same ratio exists between the corresponding changes of internal actions in general, and of other quantities which, like the deflections in general, are linear functions of the stresses. In other words, when u_m is used as a common factor defining changes of stresses, the variation of u_m alone will define a range of variations of shape; or, u_m, used as indicated, serves as a parameter of the structure. Introducing u_m as such, and summarizing the results, we have: The changes through Configurations Nos. 1, 2, and 3 may be defined by the variation of the parameter, u_m, while the remainder of the parameters retain the constant values which they have in Configuration No. 1. Configuration No. 1, the initial shape from which the changes are measured, is represented by $u_m = 0$.

The general equation of equilibrium, Formula (39), is again written:

$$L = U - P\delta = \text{min.}$$

U is a quadric in the parameters. δ is some function of the parameters, but not a linear function, because, in accordance with Article 14, a linear function would lead to orthostatic action. As $P = Q_m$ gives equilibrium throughout a continuous range of variations, it follows that δ must be a function which resembles U in some respects. It might be assumed, then, that δ is a quadratic function of the parameters, or, more exactly, that in expressing δ as a polynomial or power series, terms of higher than second degree in the parameters can be thrown away as negligible. In some cases one may verify by inspection of the structure the statement that δ is such a function.

By the following reasoning one may conclude that in the general case δ is expressed with sufficient approximation as such a quadratic function of the parameters. The function, δ, is an elastic displacement. By the assumption of rigidity made in the Introduction, the displacements are small, and they may be considered as infinitesimal quantities when compared with the main dimensions of the structure, or, the displacements may be considered as small increments to the dimensions. They can be expressed in terms of the parameters by Taylor expansions. As the increments are small, the prevailing condition, special displacements being excepted, will be that in these expansions the terms of second and higher degree are negligible when compared with the linear terms. Such displacements, therefore, are expressed with sufficient approximation as linear functions of the parameters. In the special, exceptional case of the displacement, δ, in the direction of an astatic load, P, and in some related cases, it is not possible to throw away all terms of higher than first degree in the parameters. It follows from the usual properties of expansions in the Taylor series that, in such cases, it can be expected, and it will be true in general, that a sufficient approximation is obtained by including the terms of second degree in the parameters, neglecting terms of higher degrees. On this ground it may be assumed that δ in Formula (39), in the case of astatic action, can be expressed sufficiently exactly as a quadratic

function of the parameters. In addition, however, as will be shown later, a criterion may be found for the correctness of this assumption in the very fact that astatic equilibria are known to exist; this knowledge may have been derived from other analyses or from tests.

When both U and δ are quadratic functions of the parameters, it follows that the Formulas, derived by differentiating the energy equation, Formula (39), are linear in the parameters. That is, Formulas (40),

$$\ldots\ldots \frac{\partial L}{\partial u_m} = 0 \ldots\ldots \qquad \frac{\partial L}{\partial u_n} = 0 \ldots\ldots\ldots$$

are linear in ... u_m ... u_n... also in this case of astatic action. When $P = Q_m$, the set of Formulas (40) is satisfied by the sets of values of the parameters which correspond to Configurations No. 1 and No. 2, both of which belong to the astatic equilibrium under the load, Q_m. That is, the set of Formulas (40) is satisfied by two different values of u_m, namely, $u_m = 0$, corresponding to Configuration No. 1, and u_m equal to the first chosen finite value, corresponding to Configuration No. 2. The remainder of the parameters are the same in the two cases. It follows that, in Formulas (40), all coefficients to u_m vanish when $P = Q_m$. Formulas (40), however, will be satisfied by all values of u_m as long as the remainder of the parameters are as shown in Configuration No. 1. That is, the whole range of variations of shape—defined by u_m equals any value, all other parameters being equal to definite values, namely, their values in Configuration No. 1—represents a state of astatic equilibrium produced under the critical load, Q_m. This range contains as possible cases Configurations, No. 1, No. 2, and No. 3.

We shall call u_m an astatic parameter. An astatic parameter is, therefore, by definition, a parameter which by its own variation, while the remainder of the parameters remain constant, defines the various stages of a state of astatic equilibrium.

As an example take the case of a hinged end column, of length, l, with constant cross-section, with a stiffness factor, EI, and with axial loads at the ends only. At the critical end load, $Q_m = \dfrac{\pi^2 m^2 EI}{l^2}$, an astatic equilibrium exists in which the deflected curve consists of m half waves. When x equals the distance from one end, the deflections in this buckling can be expressed as $y = u_m \sin \dfrac{m\pi x}{l}$, in which u_m may take any value. A change of u_m in this expression defines proportional changes of all stresses, curvatures, and transverse deflections. It follows that u_m, or any quantity proportional to it, can be used as an astatic parameter. In the general case of flexure, under the influence of an end load combined with transverse loads, the deflections can be expressed as $y = \Sigma u_n \sin \left(\dfrac{n\pi x}{l}\right)$, where $n = 1, 2 \ldots\ldots m \ldots\ldots$, etc. Evidently, the whole set of coefficients, $u_1 \ldots u_m$, can be used as parameters, and they will all be astatic parameters. By introducing in addition to $u_1 \ldots u_m \ldots$, a parameter which measures the axial shortening of the column, one obtains a complete set of parameters in terms of which any

possible variation of shape can be described. It follows that the astatic equilibrium under the load, Q_m, is represented by u_m, being indefinite, equal to any value, other parameters being equal to definite values, namely, some finite value for the parameter measuring the axial shortening, and the remainder being equal to zero.

The general case is now considered again. The energy Formula (39):

$$L = U - P\delta = \text{min.}$$

will be used. It has been shown that when δ is known to be quadratic, two configurations, belonging to an astatic equilibrium, determine a corresponding astatic parameter. In the discussion which follows, we may abandon temporarily the assumption that δ is known beforehand to be a function of not higher than second degree in the parameters, even though this assumption is known to represent the general case. It will be assumed only that an astatic equilibrium is known to exist, and that a corresponding parameter, u_m, exists, which is an astatic parameter in the sense of the definition just given; that is, the variation of u_m, while the rest of the parameters remain constant, represents an astatic equilibrium produced under a critical load, say, $P = Q_m$. In this astatic equilibrium, it is found, as usual, that all the first partial derivatives of L with respect to the parameters must vanish. Moreover, since u_m can vary through a continuous range of values without disturbing the condition of minimum, it follows that, also, all second and higher derivatives of L in which differentiation with respect to the astatic parameter, u_m, occurs are zero. Hence, if u_n is any other parameter, we have, when $P = Q_m$:

$$\frac{\partial^2 L}{\partial u_m^2} = 0, \qquad \frac{\partial^2 L}{\partial u_m \, \partial u_n} = 0, \ldots\ldots\ldots\ldots\ldots (47)$$

Now, assume that also u_n is an astatic parameter, and that its corresponding critical astatic load is Q_n; that is, u_n equals any value, other parameters being equal to definite values, represents an astatic equilibrium in which $P = Q_n$ is the load. Similar to Formula (47), we have then, when $P = Q_n$:

$$\frac{\partial^2 L}{\partial u_n^2} = 0, \qquad \frac{\partial^2 L}{\partial u_m \, \partial u_n} = 0, \ldots\ldots\ldots\ldots\ldots\ldots\ldots (48)$$

The resilience quadric may be written in the form:

$$U = \frac{1}{2} U_m u_m^2 + \frac{1}{2} U_n u_n^2 + U_{mn} u_m u_n + \bar{U} \ldots\ldots\ldots (49)$$

where U_m, U_n, and U_{mn} are constants and where $\bar{U}$ is a function which has no terms containing u_m^2, u_n^2, or $u_m u_n$. It follows, from the relative position of U and δ in the energy equation and from the fact that all the partial derivatives of L with respect to u_m and u_n must vanish, that δ may be written in the form:

$$\delta = \frac{1}{2} \delta_m u_m^2 + \frac{1}{2} \delta_n u_n^2 + \delta_{mn} u_m u_n + \bar{\delta} \ldots\ldots\ldots (50)$$

where δ_m, δ_n, and δ_{mn} are constants, and where $\bar{\delta}$ is a function in which terms containing u_m^2, u_n^2, $u_m u_n$, or products of higher degree in u_m and u_n, do not exist.

Substituting Formulas (49) and (50) in the expression, $L = U - P\delta$, and differentiating in accordance with Formula (47), we find:

$$U_m - Q_m \delta_m = 0 \qquad (51)$$

$$U_{mn} - Q_m \delta_{mn} = 0 \qquad (52)$$

In the same manner, differentiating according to Formula (48), we find:

$$U_n - Q_n \delta_n = 0 \qquad (53)$$

$$U_{mn} - Q_n \delta_{mn} = 0 \qquad (54)$$

Formulas (51) and (53) may be written:

$$Q_m = \frac{U_m}{\delta_m} ; \; Q_n = \frac{U_n}{\delta_n} \qquad (55)$$

These simple formulas determine the astatic loads in terms of the coefficients in the functions, U and δ.

It is assumed at present that the two critical astatic loads, Q_m and Q_n, are different. It follows that Formulas (52) and (54) can be satisfied only when

$$U_{mn} = \delta_{mn} = 0 \qquad (56)$$

that is, the mixed terms in U and δ containing the product, $u_m \, u_n$, vanish when the two parameters correspond to different critical astatic loads.

Since by Formula (44),

$$U_{mn} = \int r R_m R_n \, ds$$

we have,

$$\int r R_m R_n ds = 0 \qquad (57)$$

The vanishing of this integral represents an important property of the functions, R_m and R_n, namely, the property of orthogonality. Two functions, $\phi\,(x)$ and $\psi\,(x)$, are called orthogonal in a certain field when their product integrated over the field vanishes, that is, when $\int \phi\,(x)\,\psi\,(x)\,dx = 0$. If R_m is taken as the one function, R_n, as the other, and $rds = dx$, it is seen that Formula (57) expresses this property. When there is a whole set of orthogonal functions—as will usually be the case in astatic action—these functions become useful by their adaptability to expansions in series, particularly in problems relating to the structures in which the functions are generated. This result will be brought out by the theory of heterostatic action which follows later. The theorem expressed by Formula (57) will be referred to as the theorem of orthogonality.*

As an example, consider the case of an hinged-end column, of the length, l, with a constant stiffness factor, EI, and with $s = x =$ distance from one end. The bending moments will be used as a measure of internal action, R, that is, as

* Sets of orthogonal functions play an important part in the theory of integral equations, and, therefore, it would be expected that the subject treated here would furnish an example of application of that theory. As a matter of fact, various elements in the subject of integral equations will appear to be represented directly and physically in the problems treated. For instance, the sets of characteristic numbers, or auto-values, in the homogeneous integral equations are represented as proportional to the sets of astatic loads. Concerning the theory of integral equations, see, for example, E. T. Whittaker and G. N. Watson, "Modern Analysis", Second Edition, 1915, pp. 205 *et seq.*

generalized stresses. The bending moments in two different astatic actions may be expressed as

$$M_m = C_m \sin \frac{m\pi x}{l}, \text{ and } M_n = C_n \sin \frac{n\pi x}{l}$$

where $m \neq n$. It is well known that with $m \neq n$,

$$\int_0^l \sin \frac{m\pi x}{l} \times \sin \frac{n\pi x}{l} \times dx = 0$$

that is, with $R_m = M_m$, $R_n = M_n$, and $r = \frac{1}{EI}$, Formula (57) is satisfied.

Formula (50) shows that the part of δ which depends on the astatic parameters, u_m and u_n, alone is a quadratic function. This property is seen readily to extend to any number of astatic parameters. If in some manner or other certain parameters are known to be astatic, a corresponding part of δ will exist, which is quadratic in these parameters. Herein, is the criterion referred to previously by which, in many cases, the character of δ may be verified as a function of the parameters not higher than quadratic. This statement applies particularly to a group of cases, which will be shown to be important, namely, a group in which δ can be expressed in terms of astatic parameters, together with only one other parameter. The latter will be shown to enter into δ as a linear term; or, δ is in such cases a quadratic function of the set of parameters.

16.—Double Astatic Equilibrium.—In Article 15, the assumption was made that the two astatic loads, Q_m and Q_n, are different. The case will now be considered in which $Q_m = Q_n = Q_{mn}$. Formulas (51) to (54) are satisfied in this case when

$$Q_{mn} = \frac{U_m}{\delta_m} = \frac{U_n}{\delta_n} = \frac{U_{mn}}{\delta_{mn}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (58)$$

That is, in this special case, the mixed terms, containing u_m u_n, and having U_{mn} and δ_{mn} as coefficients, do not necessarily vanish, and the property of orthogonality is not necessarily found. Another result is that any combination of values of the two parameters, u_m and u_n, with the other parameters remaining constant, represents a possible equilibrium; that is, a double neutral or double astatic equilibrium occurs.

Some examples will illustrate this case: Consider a long cylindrical vertical column with such spherical bearings at the ends as to allow the ends to slope freely in any direction. At the lowest critical load given by the Euler formula, the column may buckle in any vertical plane through the original center line. Any such buckling may be found by superimposing a buckling curve in, say, the north-south plane on one in the east-west plane. Inversely, superposition of a buckling curve in the north-south plane on a buckling curve in the east-west plane, produced under the same critical load, leads to a possible buckling curve which will be contained in some vertical plane through the original center line. The equilibrium is a typical two-parametric astatic equilibrium, with two parameters assuming any values, while the remainder of the parameters have definite values. Another example is the thin-shelled circular cylinder carrying an outside pressure (Fig. 1 (G)). If the thickness and the material are uniform, the surface may buckle at the critical loads into

a wave line starting at any point. Since both the maximum deflection and the starting point can vary, we have again a two-parametric astatic equilibrium. The complete set of buckling surfaces at that load can be found by superimposing the buckling waves starting at two definite points. A third example is the case of the rectangular slab carrying compressive loads along its edges. It was mentioned in Article 10 that, at certain combinations of the end load, the slab may buckle into two different types of waved surfaces and into any surface found by superposition of waves of these two types.

It is evident that triple or higher-multiple astatic equilibria may also be found. In the case of an hinged-end circular column with horizontal spring supports at the ends (compare Fig. 1 (*F*)), the springs may yield and the column may buckle in two different planes at the same critical load. This would be a triple astatic equilibrium.

17.—General Formulas in Simplified Case.—Assume that astatic action can be produced under the critical astatic loads, Q_1, $Q_2 \ldots\ldots Q_n \ldots\ldots$, and let u_1, $u_2 \ldots\ldots u_n \ldots\ldots$ be the corresponding set of astatic parameters, so that, for instance, the combination, u_n = any value, other u's = 0, represents the astatic action produced when $P = Q_n$. The simplifying assumptions are made that this set of parameters is sufficient to define all the possible shapes of the structure, and that no two parameters correspond to the same Q; that is, all the Q's are different. For instance, if the axial shortenings can be neglected as being only of small influence, then the action of a slender straight column, flexible in one plane only, is described completely by an equation for deflections, of the form

$$y = \Sigma\, u_n \sin \frac{n \pi x}{l}$$

where x is the distance from one end. If the ends are hinged, and the stiffness factor, EI, is constant, then the u's in this expression are astatic parameters, and the combination, u_n = any value, other u's = 0, corresponds to the critical astatic load,

$$Q_n = \frac{n^2 \pi^2 E I}{l^2}$$

In the general case, which will now be considered, the load is assumed to consist of two component parts which may vary independently: One part is denoted by P, and is an astatic load, which, acting alone, produces astatic equilibrium when it assumes any one of the critical values, $Q_1 \ldots . Q_n . .$; the other part, W, is an orthostatic load, producing orthostatic action when acting alone. W cannot be a part of P. The combination of the two components, P and W, is a heterostatic load of a general type, producing heterostatic action. As before, δ is the displacement in the direction of P. In a similar manner, w will denote the displacement in the direction of the load, W, that is, Ww is the work of the load, W. The first object will be to find the parameters in the various possible cases. For the purpose of distinguishing between the cases, the particular values of the parameters occurring in the heterostatic action will be indicated by marking them with a prime, so that u'_n is the value of the parameter, u_n, in the heterostatic action. Without the prime,

u_n either represents the parameter as a variable quantity, or, when considered as a definite quantity, it denotes the particular value of the parameter occurring in the orthostatic action which is produced by W alone. When the parameters have been found, the various effects, such as deflections, bending moments, etc., that is, all generalized stresses, generalized strains, and generalized displacements, can be determined. These effects will be assumed to be linear homogeneous functions of the parameters. The "effects" will be denoted by F and F'. F will represent either "the effect in general", as a function of the parameters, or the particular value of the effect due to the orthostatic load, W, alone, while F' is the particular value of the effect in the heterostatic action. It is seen that the use of the prime is the same here as in the case of u_n and u_n'. Similarly, R' will denote the internal action in the heterostatic action, while R denotes either the internal action in general or the particular value of the internal action which occurs in the orthostatic action.

The notation introduced here is as follows:

Let $L =$ combined potential energy of structure and loads.

$L = 0$ when the displacement is zero.

$U =$ resilience of the structure.

$P =$ astatic component of the load.

$W =$ orthostatic component of the load.

$\delta =$ displacement in the direction of the load, W, defined by $Ww =$ work of W.

$Q_1, Q_2 \ldots\ldots Q_n \ldots\ldots =$ critical values of the astatic load, P.

$u_1 \ldots\ldots u_n \ldots\ldots =$ set of astatic parameters corresponding to the astatic load, P; in particular, when they are considered as definite quantities, they are the values of the parameters in the orthostatic action.

$u'_1 \ldots\ldots u'_n \ldots\ldots =$ same parameters in the heterostatic action.

$R =$ internal action, particularly in the orthostatic action.

$R' =$ internal action in the heterostatic action.

$ds =$ extension of structural element.

$r =$ stiffness constant, so defined that $\frac{1}{2}\, rR^2ds$ is an element of the resilience.

$F =$ effect depending on the parameters, particularly the effect in the orthostatic action. F is a linear homogeneous function of the parameters.

$F' =$ same effect in the heterostatic action.

The writer will show first how the parameters may be found in any given case, provided the internal actions are known. The intensity of internal action in any one element was expressed by Formula (42) as

$$R = R_1 u_1 + \ldots\ldots R_m u_m + \ldots\ldots R_n u_n + \ldots\ldots$$

where $R_m \ldots\ldots R_n$ are functions of the location. $u_n =$ any value, other u's being equal to zero, represents the astatic action produced by $P = Q_n$. Therefore, $R_n u_n$ is the internal action in the astatic action, Q_n, and the function,

R_n, has the particular significance of being the internal action in the astatic action, Q_n, when $u_n = 1$. For example, the bending moment, M, in Formula (11), which has reference to the general case of straight columns, may be chosen as a measure of the internal action, that is, in this case, $R = M$. The function, M_n, in Formulas (11) and (12), was defined as the bending moment in a possible buckling action, that is, we may write $R_n u_n = M_n$. Thus, Formula (11) appears as a special type of Formula (42). The general case is now considered again. Multiply Formula (42) by $rR_n ds$ and integrate so as to include all elements of energy. On account of the theorem of orthogonality, which states that $\int r\, R_m\, R_n\, d\, s = 0$, when $m \neq n$, all terms on the right side with an index different from n will vanish by the integration, and we find,

$$\int r\, R\, R_n\ \ d\, s = u_n \int r\, R_n{}^2\, d\, s$$

or,

$$u_n = \frac{\int r\, R\, R_n\, d\, s}{\int r\, R_n{}^2\, d\, s} \quad \text{.......................(59)}$$

The simplicity of Formula (59) shows the particular advantage of the sets of orthogonal functions generated in the astatic elastic actions. When the parameters have been found, their values may be substituted in the expressions for the various effects in the structure. Such effects are represented in general by a series of the form:

$$F = F_1\, u_1 + \ldots\ldots + F_m u_m + \ldots\ldots + F_n u_n \text{.........(60)}$$

where $F_m \ldots\ldots F_n$ are functions depending on the location and on the type of the effect. The internal action, R, represents a special case of the general effect, F.

The energy equation will now be discussed. In the general case, in which both loads, P and W, are acting, the energy equation takes the form:

$$L = U - P\delta - Ww = \text{min.} \text{ ....(61)}$$

the minimum here, as in Formula (39), being understood as a $\pm$ minimum. $P = 0$ gives orthostatic action, while $W = 0$ gives astatic action.

Since all the critical loads, Q, are different, the quadric, U, does not contain any products of the parameters (compare Article 15), therefore, U may be written as a series,

$$U = \frac{1}{2} \Sigma\ U_n\, u_n{}^2 \text{.........................(62)}$$

where the coefficients, U_n, are constants. It also follows that the quadratic function, δ, cannot contain products of the parameters. It will be shown a little later that δ does not contain any linear terms. δ is then a quadric similar to U and may be written:

$$\delta = \frac{1}{2} \Sigma\ \delta_n\, u_n{}^2 \text{.........................(63)}$$

where the coefficients, δ_n, are constants. It was shown in Article 14 that the displacements in the direction of orthostatic loads are linear homogeneous functions of the parameters, therefore, w may be written:

$$w = \Sigma \, w_n u_n \qquad (64)$$

where $\ldots\ldots w_n \ldots\ldots$ are constants.

The three actions, astatic, orthostatic, and heterostatic, will now be considered separately.

A.—Astatic Action.

In this case, $W = 0$. Then the energy equation takes the form:

$$L = U - P\delta = \text{min.}$$

The state of non-buckling always represents a possible equilibrium whatever the load, P, is. Hence, $u_1 = \ldots\ldots = u_n \ldots\ldots = 0$ is one possible solution of the energy equation. Linear terms in the parameters contained in δ would make some of the derivatives, $\dfrac{\partial \delta}{\partial u_n}$, contain constant terms which would prevent the vanishing of the corresponding derivatives, $\dfrac{\partial L}{\partial u_n}$, at $u_1 = \ldots\ldots u_n = \ldots\ldots = 0$. Therefore, the proposition is correct, which was mentioned previously, namely, that there cannot be any linear terms in the expression for δ. Therefore Formula (63) may be used. When $P = Q_n$, the condition, $\dfrac{\partial^2 L}{\partial u_n^2} = 0$, leads again to Formula (53), which may be written as:

$$U_n = Q_n \, \delta_n \qquad (65)$$

When this relation is satisfied, the combination $P = Q_n$, $u_n =$ any value, all other u's $= 0$, makes all the first derivatives of L, with respect to the parameters, vanish, and, therefore, this combination represents a correct solution of the energy equation.

Formula (65) allows one to express the resilience quadric in the form:

$$U = \frac{1}{2} \, \Sigma \, Q_n \delta_n u_n^2 \qquad (66)$$

B.—Orthostatic Action.

When $P = 0$, the energy equation becomes:

$$L = U - Ww = \text{min.} \qquad (67)$$

By substituting Formulas (62) and (64), and writing $\dfrac{\partial L}{\partial u_n} = 0$, we find:

$$U_n u_n - W w_n = 0 \qquad (68)$$

whereby the parameters may be determined when the coefficients in the expressions for U and w are known. On account of Formula (65), Formula (68) may also be written:

$$Q_n \delta_n u_n - W w_n = 0 \qquad (69)$$

Formulas (68) and (69) show that the parameters, u_n, are proportional to the load, W.

In a number of cases of orthostatic action, the parameters are found most conveniently by Formula (59). Since by Formula (44), $U_n = \int r\ R_n^2\ ds$, Formula (68) becomes, in fact, the same as Formula (59). The expression for w_n follows:

$$w_n = \int r\ R\ R_n\ d\ s \ldots\ldots\ldots\ldots\ldots\ldots\ldots(70)$$

Examples of the use of these methods in the analysis of orthostatic action may be found in Article 8. A part of the total resilience in the columns treated in Article 8, is due, in general, to the axial compression of the column, and this part would depend on some parameter other than the astatic parameters. A phase of the problem is involved here, which will be studied more closely in some of the subsequent articles. It will then be shown that such effects as the axial shortening do not change the applicability of the formulas and methods, just given, to the cases examined in Article 8. In the first case treated, that of hinged-end, straight columns, M_n in Formula (13) takes the place of $R_n u_n$. By choosing the coefficient, C_n, in Formula (13) as the parameter, u_n, it is found that $R_n = \sin \dfrac{n\ \pi\ x}{l}$. In this case, the factor, $r = \dfrac{1}{E\ I}$, is a constant which cancels when substituted in Formula (59). With $d\ s = d\ x$, it is found that

$$\int R_n^2\ d\ s = \int_0^l \sin^2 \frac{n\ \pi\ x}{l}\ d\ x = \frac{l}{2}$$

The substitutions, $R = M$, $u_n = C_n$, transform Formula (59) into Formula (14), or

$$C_n = \frac{2}{l} \int_0^l M \sin\ \frac{n\ \pi\ x}{l}\ d\ x$$

Formula (14), therefore, may be interpreted as a special case of the general Formula (59). Integration of Formula (14) led to the series for M given shortly after Formula (14). This series exemplifies the expressions for R and F (Formulas (42) and (60)). Also the other cases treated in Article 8 may be considered as examples of the more general theory.

C.—Heterostatic Action.

Both P and W are assumed to be different from zero. The energy equation, therefore, takes its general form, Formula (61). When U is taken from Formula (66), δ from Formula (63), and when the parameters are marked with a prime, the energy equation becomes:

$$L = \frac{1}{2}\ \Sigma\ (Q_n - P)\ \delta_n\ u'^2 - W\ \Sigma\ w_n\ u'_n = \text{min} \ldots\ldots\ldots(71)$$

Differentiation with respect to the parameter, u'_n, gives:

$$\frac{\partial\ L}{\partial\ u'_n} = (Q_n - P)\ \delta_n\ u'_n - W\ w_n = 0 \ldots\ldots\ldots\ldots\ldots(72)$$

A comparison of Formula (72) with Formula (69) leads to the following expression for the parameter, u'_n, in the heterostatic action, in terms of the

value, u_n, of the same parameter in the corresponding orthostatic action in which the orthostatic load component, W, acts alone:

$$u'_n = \frac{Q_n}{Q_n - P} u_n \qquad (73)$$

or,

$$u'_n = u_n + \frac{P}{Q_n - P} u_n \qquad (74)$$

If u_n in Formula (73) or Formula (74) is considered as a constant, then the curve, (u'_n, P), in rectangular co-ordinates becomes a hyperbola with a horizontal asymptote, $P = Q_n$. The upper branch of this hyperbola represents an unstable equilibrium. If u_n is positive, then the lower branch of the hyperbola gives positive values, and the upper branch negative values, of u'_n. The reversal of the sign indicates that the equilibrium represented by points of the upper branch can be produced only when compensating deflections have been introduced first; that is, there must be deflections opposite those which are caused by W alone and which are represented by u_n.

By substituting Formula (74) in the series, Formula (60), which expresses the "effects" in the structure, and by using the prime to designate the action as heterostatic, the following formula will be found, which expresses the effect, F', in the heterostatic action in terms of the corresponding effect, F, in the orthostatic action produced by the orthostatic load component, W, acting alone:

$$F' = F + \Sigma \frac{P}{Q_n - P} F_n u_n \qquad (75)$$

This formula furnishes a solution of the problem of heterostatic action as existing in a large group of cases. An important example of application is the general Formula (12) in Article 7, which was used extensively in Part II, and which expresses bending moments in combined column action and bending, namely,*

$$M' = M + \Sigma \frac{P}{Q_n - P} M_n$$

As referred to previously, a part of the resilience of a straight column is due to the axial shortening, but as has been mentioned in connection with an application of Formula (70), and as will be shown in the following articles, this condition does not interfere with the applicability of the general Formula (75) to the case represented by Formula (12). One derives Formula (12) from Formula (75), by taking the bending moments, M and M', as the "effects", F and F', in Formula (75), and by substituting the bending moment, M_n, which is produced in the buckling under the axial load, Q_n, for the effect, $F_n u_n$, which is produced in the general astatic action under the load, Q_n.

* Goupil in *Annales des Ponts et Chaussées*, 1912, V, p. 401, indicates a formula and a rule of computation which might be derived as another special case of Formula (75). The case is that of a circular ring with constant cross-section, or of a cylinder with constant thickness. The load consists of pressures from the inside or outside, distributed in any manner. The "effects" are the deflections, but it is also shown how the bending moments may be found. The formula is similar to Formulas (75) and (12). It is evident that Formula (75), in the form of Formula (12), applies not only to straight, but also to curved members. Goupil quotes this derivation as having been indicated in lectures by Marbec and Le Besnerais at l'Ecole du Génie Maritime. This derivation is based on the differential equation of flexure applying in the particular case, and use is made of the Fourier series.

18.—Independent Parameters.—We shall now leave the simplified case treated in Article 17 and consider a case of a general nature. Assume that P denotes any load acting on the structure, and that no other loads are applied. Then the energy equation takes the form of Formula (39), or,

$$L = U - P\delta = \text{min.}$$

Assume that the resilience, U, can be expressed:

$$U = \bar{U}\ (u_1 \ldots\ldots u_n \ldots\ldots) + T\ (t_1 \ldots\ldots t_i \ldots\ldots) \ldots\ldots\ldots (76)$$

where $\bar{U}$ is a quadric in $u_1 \ldots\ldots u_n \ldots\ldots$, and T a quadric in $t_1 \ldots\ldots t_i \ldots\ldots$ As $\bar{U}$ cannot be negative, neither U nor T can be negative. It is assumed that $T = 0$ only when $t_1 = \ldots\ldots = t_i = \ldots\ldots = 0$. The displacement, δ, in the direction, P, is in the general case a quadratic function of the parameters. Assume that δ depends on the u's only, or,

$$\delta = \delta\ (u_1 \ldots\ldots u_n \ldots\ldots)$$

Under these circumstances, the general energy equation, Formula (39), is equivalent to the two independent conditions combined, namely,

$$T = \text{min.} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (77)$$

and,

$$L' = \bar{U} - P\delta = \text{min.} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (78)$$

Under the assumptions made, the only solution of Formula (77) is $t_1 = \ldots\ldots = t_i = \ldots\ldots = 0$. Formula (78) is an energy equation of the usual type shown in Formula (39), but contains only the parameters, $u_1 \ldots\ldots u_n$. Under these conditions, the parameters, $t_1 \ldots\ldots t_i$, are "parameters independent of the load, P". If P is an astatic load, or the astatic component of a load of general type, the parameters, $t_1 \ldots\ldots t_i$, will be referred to simply as "independent parameters".

When P is an astatic load, the occurrence of independent parameters may be represented as a limiting case in which one or more of the coefficients, δ_m, δ_n, $\delta_{mn} \ldots$, in the expressions for δ, such as Formula (50) or Formula (63), become equal to zero without the vanishing of the corresponding coefficients in U. When u_m is an astatic parameter, the corresponding astatic load is expressed, in general, as $Q_m = \dfrac{U_m}{\delta_m}$ (Formula (55)). Hence, $\delta_m = 0$ represents the limiting case, $Q_m = \infty$, the case in which the astatic load, Q_m, has become infinite. In the same manner, $\delta_m = \delta_n = \delta_{mn} = 0$ represents the limiting case, $Q_m = Q_n = \infty$.

As an example, consider an hinged-end column which is originally vertical and is supported at the ends by two horizontal springs, as indicated in Fig. 1(F). A horizontal parallel motion of the column would not change the vertical distance between the ends, therefore, a parameter expressing such a variation of position is an independent parameter. When the two springs are equally stiff, this parameter can be measured in any state of deformations, as the sum of the horizontal displacements of the two ends, considered positive in the same direction. Another parameter which is measured by the difference of the horizontal deflections of the ends, is a regular astatic parameter. The

functions which represent the internal actions corresponding to those two parameters are easily shown to be orthogonal, as they should be, because one corresponds to an infinite, and the other to a finite, critical astatic load.

As another example, take a structure consisting of a column plus some independent structure. The parameters defining the variations in this independent structure are independent parameters with reference to the axial load on the column.

19.—The Orthostatic Parameter in Pure Astatic Action, and the Vanishing of Certain Constants.—Consider the vertical hinged-end column, of a length, l, with a uniform stiffness factor, EI, an area of cross-section, A, and with the axial loads transferred at the ends only. It was pointed out in Article 17 that the coefficients, u_n, in the expression for the deflections, $y = \Sigma\, u_n \sin \frac{n \pi x}{l}$, combined with one parameter measuring the axial shortening, would form a complete set of parameters in terms of which any possible type of deformation could be expressed. At the beginning of Article 17, it was stated that if the column is very slender, the axial shortening would have only a negligible influence on the total energy variations, and in that case the column would belong directly in the simplified class treated in Article 17. It was also stated—without proof—that certain applications of the formulas derived could be made even if an appreciable amount of energy is consumed by the axial shortening. The questions involved here will now be investigated.

Let v denote the parameter which measures the axial shortening; that is, v is a quantity proportional to the uniformly distributed axial shortening due to the direct compressive stress. When the terms depending on v have been added, the resilience, U, and the displacement, δ, in the direction of P, are expressed as follows:

$$U = \frac{1}{2}\, \Sigma\, U_n\, u_n^2 + \frac{1}{2}\, V\, v^2$$

$$\delta = \frac{1}{2}\, \Sigma\, \delta_n\, u_n^2 + \gamma\, v$$

in which V and γ are constants. The energy equation, $L = U - P\,\delta = \text{min.}$, gives, in this case, $\frac{\partial L}{\partial v} = V\, v - P\,\gamma = 0$, or, $v = \frac{P\gamma}{V}$, that is, v is proportional to the load.

Also, in other cases of astatic action, there is generally one variation of shape which is directly proportional to the load and which can be expressed by a parameter such as v. In the special solution in which all the astatic parameters are zero, the deformations are, as a whole, proportional to v and thereby to the astatic load, P. Such a parameter, which measures that particular variation in the astatic action which is proportional to the load, will be called an orthostatic parameter.

A general case will be considered in which the variations of the structure can be expressed in terms of the following three kinds of parameters: Astatic parameters, $u_1 \ldots\ldots u_n \ldots\ldots$; the orthostatic parameter, v; and the independent parameters, $t_1 \ldots\ldots t_i \ldots\ldots$ When these parameters are sufficient to define all the possible variations of the structure, the astatic action produced by the astatic

load, P, corresponding to the parameters, $u_1 \ldots\ldots u_n$, will be called a pure astatic action. This action is characterized by the condition that $P = Q_n$ gives as a possible solution, $u_n =$ any value, v being proportional to Q_n and all the other parameters being zero.

The energy equation, Formula (39), will be used again,

$$L = U - P\delta = \min.$$

In this case, we write:

$$U = \overline{U}(u_1 \ldots\ldots u_n \ldots) + \frac{1}{2} V v^2 + v \Sigma V_n u_n + T(t_1 \ldots\ldots t_i \ldots\ldots) \ldots\ldots (79)$$

and,

$$\delta = \overline{\delta}(u_1 \ldots\ldots u_n \ldots) + \Sigma \varepsilon_n u_n + \gamma v + v \Sigma \gamma_n u_n \ldots\ldots\ldots (80)$$

in which $\overline{U}$, T and $\overline{\delta}$ are quadrics, while V, V_n, ε_n, γ, and γ_n are constants. Differentiation gives:

$$\frac{\partial L}{\partial u_n} = \left(\frac{\partial \overline{U}}{\partial u_n} - P \frac{\partial \overline{\delta}}{\partial u_n}\right) - P \varepsilon_n + (V_n - P \gamma_n) v = 0 \ldots\ldots (81)$$

and,

$$\frac{\partial L}{\partial v} = (V v - P \gamma) + \Sigma (V_n - P \gamma_n) u_n = 0 \ldots\ldots\ldots\ldots (82)$$

Since $u_1 = \ldots\ldots u_n = \ldots\ldots = 0$ is a possible solution at any value of P, Formula (82) gives $V v - P \gamma = 0$, or,

$$v = \frac{P \gamma}{V} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (83)$$

$P = Q_n$ gives as a possible solution $u_n =$ any value, other u's being zero, therefore, Formula (82) gives $V_n = Q_n \gamma_n$. All u's being equal to zero makes $\frac{\partial \overline{U}}{\partial u_n} - P \frac{\partial \overline{\delta}}{\partial u_n} = 0$, that is, in this solution the first term in Formula (81) vanishes. When, especially, $P = Q_n$, the last term in Formula (81) vanishes, therefore, $Q_n \varepsilon_n = 0$, that is,

$$\varepsilon_1 = \ldots\ldots\ldots\ldots = \varepsilon_n = \ldots\ldots\ldots = 0 \ldots\ldots\ldots\ldots\ldots (84)$$

Finally, the combination, $u_1 = \ldots\ldots u_n = \ldots\ldots = 0$, v being finite, $P \neq Q_n$, gives, on account of Formula (81):

$$V_n = \gamma_n = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (85)$$

Formulas (84) and (85) express the result that, in the case of pure astatic action, the linear terms in the astatic parameters vanish in the expression for δ, and the mixed terms containing products of the orthostatic and the astatic parameters vanish in the expression for U and δ.

20.—The Parameters and the Energy Equation in the General Case.—A case will now be treated which is more general than the simplified one analyzed in Article 17. As in Article 17, the load will be assumed to consist of two component parts: $W =$ orthostatic component of the load, and $P =$ astatic component of the load.

As before, it is assumed that the series, $Q_1, Q_2 \ldots\ldots Q_n \ldots\ldots$, is the series of different critical values of the astatic load, P. It is also assumed that some

finite number of astatic parameters corresponds to each critical load, so that the complete set of astatic parameters can be written:

u_{1a}, u_{1b}.................u_{1k}, ...corresponding to Q_1;
u_{2a}, u_{2b}.................u_{2k}, ...corresponding to Q_2;
....
u_{na}, u_{nb}.................u_{nk}, ...corresponding to Q_n.
....

In addition, there are the following special parameters: v = orthostatic parameter; and $t_1, t_2 \ldots\ldots t_i \ldots\ldots$ = independent parameters.

It is assumed that these parameters are sufficient to determine the possible variations of shape of the structure. It follows then from the definition given in Article 19 that the buckling under the influence of P is a pure astatic action. A prime will be used again to denote the particular value of a parameter in the heterostatic action; so that we have, for instance, u'_{nk} = the particular value of the parameter, u_{nk}, in the heterostatic action. The notation, δ, w, R, R', ds, r, F, and F', is the same as that given in Article 16.

The energy equation takes again the form of Formula (61), or,

$$L = U - P\delta - Ww = \text{min.}$$

When writing expressions for U and δ use will be made of the theorem of the vanishing of the mixed terms which contain the products of two astatic parameters that belong to different critical loads (Article 15). The coefficients of certain other terms also vanish, as was indicated in Article 19. The result is that the resilience, U, and the displacements, δ and w, can be expressed as follows:

$$U = \Sigma U^{(n)} + \frac{1}{2} Vv^2 + T(t_1, t_2 \ldots) \qquad \text{(86)}$$

$$\delta = \Sigma \delta^{(n)} + \gamma v \qquad \text{(87)}$$

$$w = \sum_n \sum_i w_{ni} u_{ni} + \bar{w} \qquad \text{(88)}$$

in which $U^{(n)}$ and $\delta^{(n)}$ are quadrics in the parameters, $u_{na} \ldots\ldots u_{ni} \ldots\ldots$, which belong to the critical load, Q_n, and in which

V, γ, w_{ni} = constants;
T = quadric in the parameters, $t_1, t_2 \ldots\ldots$; and
$\bar{w}$ = linear function of $t_1, t_2 \ldots\ldots$, and v.

The quadric $\delta^{(n)}$ may be written:

$$\delta^{(n)} = \frac{1}{2} \sum_i \delta_{ni} u_{ni}^2 + \Sigma \delta_{ni,nk} u_{ni} u_{nk} \qquad \text{(89)}$$

The relation, Formula (58), between the coefficients in U and δ applies to the coefficients in Formula (89), therefore, the quadric, $U^{(n)}$, can be written in the form:

$$U^{(n)} = Q_n \, \delta_n \qquad \text{(90)}$$

21.—Solution of the Energy Equation in the General Case.—

A.—Astatic Action.

Assume $P = Q_n$, $W = 0$, then the energy equation, Formula (61), is satisfied when $u_{na}, u_{nb} \ldots\ldots\ldots u_{ni} \ldots\ldots$ assume any values, and $v = P\,\frac{\gamma}{V}$ (Formula 83), all other parameters being equal to zero.

B.—Orthostatic Action.

$P = 0$ puts the energy equation in the form $L = U - Ww = \text{min}$. Differentiation then gives a set of equations of the type:

$$\frac{\partial L}{\partial u_{ni}} = Q_n \frac{\partial \delta^{(n)}}{\partial u_{ni}} - Ww_{ni} = 0 \ldots\ldots\ldots\ldots\ldots\ldots (91)$$

or, if Formula (89) is used:

$$Q_n\,(\delta_{na,ni}\;u_{na} + \delta_{nb,ni}\,u_{nb} + \ldots + \delta_{ni}\,u_{ni} + \ldots) = Ww_{ni} \ldots\ldots (92)$$

By making $i = 1, 2, 3 \ldots\ldots$, a set of equations similar to Formula (92) is obtained. One such set of linear equations is derived for each critical load, and thereby sufficient relations are found for the determination of all the astatic parameters. In the special case in which there is only one astatic parameter, u_n, corresponding to Q_n, the quadric, $\delta^{(n)}$, is reduced to the form:

$$\delta^{(n)} = \frac{1}{2}\;\delta_n u^2_n$$

whereby Formula (92) is reduced to the following special form, which is equivalent to Formula (69):

$$Q_n\;\delta_n u_n = Ww_n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (93)$$

The orthostatic parameter, v, and the independent parameters, t_1, $t_2 \ldots\ldots$, are found in a similar manner from equations of the form:

$$\frac{\partial L}{\partial v} = 0, \frac{\partial L}{\partial t_1} = 0, \frac{\partial L}{\partial t_2} = 0, \text{etc.}$$

C.—Heterostatic Action.

In this case, the astatic parameters are denoted with a prime, as, for example, u'_{ni}. By substituting Formulas (86), (87), (88), and (90), in the energy equation, $L = U - P\delta - Ww = \text{min.}$, and differentiating with respect to the astatic parameters, equations of the following type are found:

$$\frac{\partial L}{\partial u'_{ni}} = (Q_n - P)\,\frac{\partial \delta^{(n)}}{\partial u'_{ni}} - Ww_{ni} = 0 \ldots\ldots\ldots\ldots\ldots (94)$$

These equations are linear. By the substitution of Formula (89) for $\delta^{(n)}$, Formula (94) is transformed into:

$$(Q_n - P)\,(\delta_{na,ni}\,u'_{na} + \delta_{nb,ni}\,u'_{nb} + \ldots + \delta_{ni}\,u'_{ni} + \ldots) = Ww_{ni} \ldots (95)$$

A set of equations like Formula (95) is obtained by making $i = 1, 2, 3 \ldots\ldots$ One such set of equations belongs to each astatic load. By comparing Formula (95) with Formula (92), the following general relation is found between the values, u_{ni}, of the parameters in the orthostatic action produced by W alone,

and the values, u'_{ni}, of the same parameters in the heterostatic action produced by P and W together:

$$u'_{ni} = \frac{Q_n}{Q_n - P} u_{ni} \qquad (96)$$

or,

$$u'_{ni} = u_{ni} + \frac{P}{Q_n - P} u_{ni} \qquad (97)$$

These formulas are actually the same as Formulas (73) and (74), which were derived for the simplified case. That is, Formulas (73) and (74) can be applied generally.

The orthostatic parameter in the astatic action was found to be $v = \frac{P\,\gamma}{V}$ (Formula (83)). Assume that the orthostatic action gives the value, v_0. Since v enters only in linear terms in δ and w (Formulas (87) and (88)), the value, v', of the orthostatic parameter in the heterostatic action can be found by adding the values in the corresponding astatic and orthostatic actions, that is,

$$v' = \frac{P\,\gamma}{V} + v_0 \qquad (98)$$

The derivatives, $\frac{\delta L}{\delta t_1}, \frac{\delta L}{\delta t_2}$, etc., are the same as those in the orthostatic action, hence the independent parameters, $t_1, t_2, \ldots\ldots$, are the same in the heterostatic action as in the corresponding orthostatic action.

Now, consider some effect, F. The particular value of F in the heterostatic action is denoted by F'. Assume that F can be expressed as,

$$F = \Sigma\, F^{(n)} + F_0\, v + \bar{F}\,(t_1, t_2 \ldots) \qquad (99)$$

where F_0 is a constant and where $F^{(n)}$ is a linear homogeneous function of the astatic parameters belonging to Q_n. That is, $F^{(n)}$ can be written:

$$F^{(n)} = F_{na}\, u_{na} + F_{nb}\, u_{nb} + \ldots \quad + F_{ni}\, u_{ni} + \ldots \qquad (100)$$

We now introduce the particular values, $\ldots u'_{ni}$ and v', given by Formulas (97) and (98), in Formulas (99) and (100), and we find thereby the value F' of the effect F in the heterostatic action. This value, F', is then expressed as follows in terms of the corresponding values, F, u_{ni} and $F^{(n)}$, which occur in the orthostatic action:

$$F' = F + \sum_n \frac{P}{Q_n - P} \sum_i F_{ni}\, u_{ni} + P\,\frac{F_0\,\gamma}{V} \qquad (101)$$

or,

$$F' = F + \sum_n \frac{P}{Q_n - P} F^{(n)} + P\,\frac{F_0\,\gamma}{V} \qquad (102)$$

Formulas (101) and (102) express the effect in the heterostatic action in a case of a most general character, namely, the general case in which the heterostatic load can be resolved into one orthostatic component, W, and one component, P, giving pure astatic action. Like F and u_n in Formula (75) (which is the general formula derived in the simplified case), F, u_n, and $F^{(n)}$ in Formulas (101) and (102) are values belonging to the "corresponding ortho-

static action" produced when $P = 0$. Formula (75) may be derived as the special case of Formula (102) in which $F^{(n)} = F_n\, u_n$ and $F_0\, \gamma = 0$. The general Formula (12) used in Part II is seen readily to be the special case of Formula (102), in which the "effects", F, F', and $F^{(n)}$, are the bending moments, M, M', and M_n, respectively. F_0 is zero in that special case. These considerations also prove finally that the applications made in Article 17 to straight columns are valid whether or not the axial shortening is appreciable.

22.—Determination of the Parameters in the General Case, When the Internal Actions Are Known.—The intensity of internal action, the generalized stress, R, may be considered as an "effect", and may be expressed in the same manner as F in Formula (99). We write then:

$$R = \Sigma\, R^{(n)} + R_0\, v + R\, (t_1, t_2 \ldots\ ) \ldots\ldots\ldots\ldots\ldots (103)$$

in which,

$$R^{(n)} = R_{na}\, u_{na} + R_{nb}\, u_{nb} + \ldots\ + R_{ni}\, u_{ni} + \ldots\ldots\ldots\ldots (104)$$

The euergy of the element, ds, is $\frac{1}{2}\, r R^2 ds$, so that the total resilience is $U = \frac{1}{2} \int r R^2 ds$ (Formula (41)). U is a quadric in the parameters, but has no terms containing products of the following types: $u_{ni} u_{mk}$, where $n \neq m$; $u_{ni}\, v$; $u_{ni}\, t_k$; $v t_k$. This property of u is expressed in Formula (86), which omits all such terms. It follows that the property of orthogonality, proved in Article 15 with reference to two astatic parameters, is extended so as to include the following relations:

$$\int r R_{ni}\, R_{mk}\, ds = 0 \quad (m \neq n, \text{ compare Formula (57)}) \ldots\ldots\ldots (105)$$

$$\int r R_{ni}\, R_0\, ds = 0\,; \int r R_{ni}\, \bar{R} ds = 0\,; \int r R_0\, \bar{R} ds = 0 \ldots\ldots (106)$$

Multiplying Formula (103) by $r R_{ni}\, ds$ and integrating so as to include all elements of energy, thereby making use of Formulas (105) and (106), we find,

$$\int r\, R\, R_{ni}\, d\, s = \int r\, R^{(n)}\, R_{ni}\, ds \ldots\ldots\ldots\ldots\ldots (107)$$

or,

$$\int r\, R\, R_{ni}\, ds = u_{na} \int r\, R_{na}\, R_{ni}\, ds + u_{nb} \int r\, R_{nb}\, R_{ni}\, ds + \ldots\ldots$$
$$+\, u_{ni} \int r\, R_{ni}{}^2\, ds + \ldots\ldots\ldots\ldots\ldots\ldots (108)$$

The latter equation (Formula (108)) is one of a set which may be written:

$$\left.\begin{aligned}
&u_{na} \int r\, R_{na}{}^2\, ds + u_{nb} \int r\, R_{nb}\, R_{na}\, ds + \ldots\ldots \\
&\quad + u_{ni} \int r\, R_{ni}\, R_{na}\, ds + \ldots\ldots = \int r R\, R_{na}\, ds \\
&u_{na} \int r\, R_{na}\, R_{nb}\, ds + u_{nb} \int r\, R_{nb}{}^2\, ds + \ldots\ldots \\
&\quad + u_{ni} \int r\, R_{ni}\, R_{nb}\, ds + \ldots\ldots = \int r R\, R_{nb}\, ds \\
&\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots
\end{aligned}\right\} \ldots (109)$$

This set contains one equation for each of the parameters, $u_{na}, u_{nb}, \ldots u_{ni} \ldots$, which belong to the critical load, Q_n. One set of such equations can be derived for each different critical load, and thereby a number of equations is found, which is sufficient for the determination of all the astatic parameters. A very important special case is that in which u_{na} is the only parameter belonging to the critical load, Q_n. In that case the set, Formula (109), is reduced to one equation, which may be written:

$$u_{na} = \frac{\int r\, R\, R_{na}\, ds}{\int r\, R_{na}^2\, ds} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (110)$$

The index, n, may be used in this special case instead of na. Then, Formula (110) becomes identical with Formula (59), which was derived for the simplified case in Article 17, in a very similar manner.

Formula (110) applies under special circumstances, even when there is more than one astatic parameter belonging to Q_n. It applies whenever the condition, $\int r\, R_{nb}\, R_{na}\, d\, s = \ldots. = \int r\, R_{ni}\, R_{na}\, d\, s = \ldots. = 0$, is satisfied. In such a case, the parameter, u_{na}, will be said to have been orthogonalized with respect to the other parameters in the group, $u_{nb} \ldots . u_{ni}$. It is always possible, by a proper choice of the parameters in the group belonging to one critical load, to orthogonalize all the parameters in this group with respect to each other. This process is often facilitated by properties of symmetry of the structure. Take, for instance, the vertical hinged-end column with spherical bearings and with circular cross-section: Two astatic parameters will correspond to each critical load. Each such pair of parameters is orthogonalized by choosing the parameters so that they measure the amounts of buckling in two vertical planes perpendicular to one another.

The orthostatic parameter, v, is found in a similar way. Multiplying Formula (103) by $r\, R_0\, ds$, integrating, thereby using the properties of orthogonality expressed in Formula (106), and solving for v, we find an expression, similar to Formula (110), namely,

$$v = \frac{\int r\, R\, R_0\, d\, s}{\int r\, R_0^2\, d\, s} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (111)$$

The independent parameters, $t_1, t_2 \ldots\ldots$, can be determined in the same manner as any group of astatic parameters. One finds a set of equations of the same form as that shown in Formula (109).

23.—Mixed Astatic Action.—We will consider again the general formula for heterostatic action, namely,

$$F' = F + \sum_n \frac{P}{Q_n - P}\, F^{(n)} + P\, \frac{F_0\, \gamma}{V} \quad \ldots\ldots\ldots\ldots \quad (102)$$

where $F^{(n)} = \Sigma\, F_{ni}\, u_{ni}$. In the special case, $P = Q_n$, the effect, F', would generally become infinite, except when $F^{(n)} = 0$, in which particular case, Formula (102) indicates F' as indefinite. This consideration suggests that P combined with special types of the orthostatic load, W, may cause astatic actions which differ from the pure astatic action produced by P acting alone, but which have, nevertheless, some of the characteristic features of pure buckling. The most important of these features is that of the indefinite dis-

placements at constant values of the load. The possibility of such "mixed astatic actions", produced under the combined influence of a pure astatic load and an orthostatic load, will now be examined.

Consider Formula (95): It is one of a set belonging to Q_n. The set is completed by exchanging the indices, *na, nb......*, respectively, for *ni*. It is assumed that,

$$w_{na} = w_{nb} = \ldots\ldots = w_{ni} = \ldots\ldots = 0.$$

Then the set of Formulas (95) is solved as follows:

$$\text{When } P \neq Q_n, u'_{na} = u'_{nb} = \ldots\ldots u'_{ni} = \ldots\ldots = 0;$$
$$\text{When } P = Q_n, u'_{na}, u'_{nb} \ldots\ldots u'_{ni} = \ldots\ldots = \text{any values.}$$

This solution satisfies Formula (94), and when the other parameters have been found by the methods indicated in Article 21, by using, for instance, Formulas (97) and (98), the combined solution will satisfy the general energy equation, Formula (61). The conclusion is that $P = Q_n$ produces astatic action when combined with W, provided the coefficients, $w_{na}\ldots\ldots w_{ni}\ldots\ldots$, vanish. On account of Formula (92), this condition can also be expressed as follows: Astatic action is possible when the values, $u_{na}, u_{nb}\ldots\ldots u_{ni}\ldots\ldots$, as determined in the orthostatic action produced by W alone, are zero.

A whole series of special critical loads, $Q_n, Q_n', Q_n''\ldots\ldots$, forming a part of the complete series of critical values of P, may give astatic action, although the astatic load is combined with the orthostatic load, W. Now, assume that W varies proportionally to P. In that case, the combination, P, W, can be considered as one load, the intensity of which is measured by P. As it produces astatic action when the intensity reaches the critical values, $Q_n, Q_n', Q_n''\ldots\ldots$, the heterostatic load, P, W, may be considered as an astatic load. Let u_{mk} be some astatic parameter which W, acting alone, causes to be different from zero, and let the value, u_{mk}, be the particular value produced by W alone; then, Formula (97) would give the corresponding value of the parameter produced by the combination, P, W, namely,

$$u'_{mk} = u_{mk} + \frac{P}{Q_m - P} u_{mk}$$

u_{mk} is proportional to W, which is assumed to be proportional to P. It follows that u'_{mk} is neither constantly zero, nor proportional to P, and therein lies a principal difference between the pure astatic action produced by P, and astatic action produced by the load, P, W. It is the latter type of astatic action which is designated as "mixed astatic action". This discussion may be summarized as follows: The load, P, W, in which W is proportional to P, may produce astatic action at a series of critical values of P. When one or more of the astatic parameters becomes different from zero, under the influence of W, then stresses and other effects in the structure are neither all zero, nor proportional to the load, even when the load is different from any critical value. In that case, the action is a mixed astatic action, to be distinguished from the pure astatic action in which, as long as the load is different from its critical values, the effects are either zero or proportional to the load.

In the case analyzed, $u_{na}, u_{nb}, \ldots\ldots u_{ni}\ldots\ldots$ are astatic parameters, not only with reference to the load, P, but also with reference to the astatic load, P, W.

Yet, u_{mk} can be classified neither as an astatic, an orthostatic, nor as an independent parameter. The only case in which the effects in a structure can be described completely in terms of astatic parameters, one orthostatic parameter, and independent parameters, with reference to a certain load, is that in which this load produces pure astatic action. In most cases, it is possible to conclude from the nature of the structure and from the nature of the load whether a certain astatic action is a pure or a mixed action. If it is a mixed astatic action, then, in order to make the general Formulas (101) and (102) applicable, the load should be resolved into two components, one, P, giving pure astatic action, the other, W, giving orthostatic action.

As an example, consider a hinged-end column with the load, P, transferred at the ends with equal but opposite eccentricities, $\pm e$. The case was treated in Article 9. Astatic equilibria occur when $P = \frac{n^2 \pi^2 E I}{l^2}$, where $E I =$ the stiffness factor, $l =$ the length, and $n = 1, 3, 5 \ldots\ldots$ Between these critical loads, the stresses, deflections, and other actions do not increase proportionally to the load, except at special points. The case may be classified as heterostatic action or as mixed astatic action. The load may be resolved into an astatic component giving pure astatic action, namely, a central end load, P, and an orthostatic component, namely, the end moments, Pe, both clockwise, or both anti-clockwise. If the same column is loaded by a central axial end load and, at the same time, by a symmetrical transverse load, then astatic equilibria occur at $P = \frac{n^2 \pi^2 E I}{l^2}$, with $n = 2, 4, 6 \ldots\ldots$ Also, this case is evidently one of mixed astatic action.

24.—Initial Eccentricities in a Simplified Case.—A structure is considered, which depends on the parameters, $u_1 \ldots\ldots u_n$, which are all astatic parameters with respect to the load, P. When all the critical loads, $Q_1 \ldots\ldots Q_n$, are different, then U and δ of the energy equation, $L = U - P\,\delta =$ min., can be expressed as in Formulas (66) and (63) in Article 17,

$$U = \frac{1}{2}\ \Sigma\ Q_n\,\delta_n\,u_n^{\,2}\,; \qquad \delta = \frac{1}{2}\ \Sigma\ \delta_n\,u_n^{\,2}$$

Now, assume a change of shape defined by $u_1 = e_1, \ldots\ldots u_n = e_n \ldots$, and assume that these deformations remain as a "permanent set" after the release of the load. It is also assumed that zero stresses correspond to zero load after the change has taken place, and that all other elastic properties, stiffness constants, etc., remain as before the change. In the changed structure, the original load, P, will no longer produce pure astatic action. The new case can be described as differing from the original case by eccentricities which are measured by $e_1 \ldots\ldots e_n \ldots\ldots$ By definition, $e_1 \ldots\ldots e_n \ldots\ldots$ will be called "the eccentricities", or, more explicitly, "the eccentricities of the parameters, $u_1 \ldots\ldots u_n \ldots\ldots$, with respect to the astatic load, P". The two structures, the original and the changed, will be called the "concentric" and the "eccentric" structures, respectively.

The quantities, $u_1 \ldots\ldots u_n \ldots\ldots$, can be used as parameters of both the eccentric and the concentric structure. The method of measuring the param-

eters starting from the initial shape of the concentric structure, will first be used; that is, in the eccentric structure, $u_1 = e_1, \ldots\ldots u_n = e_n \ldots\ldots$, represents the state of zero stresses, while $u_1 = \ldots = u_n = \ldots = 0$ indicates the shape which coincides with the initial shape of the concentric structure. This is one of the exceptional cases, in which the general rule laid down in Article 11, that zero values of all the parameters represent zero stresses, is not followed. Since the stiffness constants are the same in both structures, the resilience quadric of the eccentric structure becomes:

$$U = \frac{1}{2} \Sigma \, Q_n \, \delta_n \, (u_n - e_n)^2 \ldots\ldots\ldots\ldots\ldots\ldots (112)$$

When δ is measured, starting from the initial shape of the original structure, the expression, Formula (63), for δ remains unchanged, that is, we have also in the eccentric structure:

$$\delta = \frac{1}{2} \Sigma \, \delta_n \, u_n^2$$

Substituting Formulas (63) and (112) in the equation, $L = U - P\delta = \text{min.}$, one finds the energy equation of the eccentric structure:

$$L' = \frac{1}{2} \Sigma \, (Q_n - P) \, \delta_n \, u_n^2 - \Sigma \, Q_n \, \delta_n \, e_n \, u_n = \text{min} \ldots\ldots\ldots (113)$$

Differentiation with respect to u_n gives:

$$u_n = e_n + \frac{P}{Q_n - P} \, e_n \ldots\ldots\ldots\ldots\ldots\ldots (114)$$

By comparing Formula (114) with the general Formula (74) derived in Article 17, it is seen that the value of the parameter, u_n, in the eccentric action can be found as the value of the same parameter in a certain heterostatic action which occurs in the original concentric structure, namely, the heterostatic action in which the astatic load, P, is combined with an orthostatic load which, if acting alone, would produce the initial shape of the eccentric structure. The character of this orthostatic load, W, and of its displacement, w, is defined by Formula (115):

$$W \, w = \Sigma \, Q_n \, \delta_n \, e_n \, u_n \ldots\ldots\ldots\ldots\ldots\ldots (115)$$

Effects which are measured, starting from the initial shape of the original structure, such as deflections from this initial shape, can be found by substituting the values of $u_1 \ldots\ldots u_n \ldots\ldots$, found by Formula (114), in the formulas which determine these effects in the original structure. Other effects, such as stresses or deflections from the initial shape of the eccentric structure, effects which are zero when $u_1 = e_1, \ldots\ldots u_n = e_n \ldots\ldots$, are found by substituting $u_1 - e_1, \ldots\ldots u_n - e_n \ldots\ldots$ for $u_1 \ldots\ldots u_n \ldots\ldots$ in the formulas for these effects in the original structure.

This result suggests that, in certain cases, it might be advantageous to measure the parameters starting not from the initial shape of the original structure, but from the initial shape of the eccentric structure. Let the parameters measured in this manner be $\bar{u}_1 \ldots\ldots \bar{u}_n \ldots\ldots$ They are derived from the parameters, $u_1 \ldots\ldots u_n \ldots\ldots$, simply by subtracting the eccentricities,

$e_1 \ldots\ldots e_n \ldots\ldots$; that is, $\ldots\ldots \bar{u}_n = u_n - e_n$, etc. Hence the resilience is expressed as follows:

$$U = \frac{1}{2} \Sigma\, Q_n\, \delta_n\, \bar{u}_n^{\,2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (116)$$

By measuring δ as before, from the initial shape of the original structure, it is found that,

$$\delta = \frac{1}{2} \Sigma\, \delta_n\, (\bar{u}_n + e_n)^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (117)$$

Substitution of Formulas (116) and (117) puts the energy equation, $L = U - P\delta = \text{min.}$, in the form:

$$L'' = \frac{1}{2} \Sigma \left(Q_n - P \right) \delta_n \bar{u}_n^{\,2} - P\, \Sigma\, \delta_n e_n \bar{u}_n = \text{min.} \ldots\ldots (118)$$

Differentiation with respect to $\bar{u}_n$ gives:

$$\bar{u}_n = \frac{P}{Q_n - P}\, e_n \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (119)$$

Since $u_n = \bar{u}_n + e_n$, Formula (119) is actually the same as Formula (114). Formula (118) leads to another interpretation of the eccentric action as a case of heterostatic action, and appears as the equation of a heterostatic action in which both the orthostatic and the astatic components of the load are measured in intensity by P. The orthostatic component is defined by its work, $Ww = P \Sigma\, \delta_n e_n \bar{u}_n$. The displacement in the direction of the astatic load in Formula (118) is expressed as $\delta' = \frac{1}{2} \Sigma\, \delta_n \bar{u}_n^{\,2}$, while Formula (117) expresses the displacement in the direction of the original load. The astatic component in Formula (118) may be obtained by adding certain loads proportional to P to the original load. Since the total given load, P, is the heterostatic load in Formula (118), however, the loads mentioned (which were to be added to the given load in order to produce the astatic load) will be equal and opposite to the loads which constitute the orthostatic component.

25.—Initial Eccentricities in the General Case.—Consider a structure of a more general type, using the notation given in Article 20. A load, P, is assumed to produce pure astatic action. A change or deformation is considered, in which the astatic parameters, $u_{na} \ldots\ldots u_{ni} \ldots\ldots$, belonging to the critical load, Q_n, take the values $e_{na} \ldots\ldots e_{ni} \ldots\ldots$, and in which the other astatic parameters take values which are denoted correspondingly. These deformations are assumed to be retained as a permanent set after the release of the load, so that the changed shape is now that which corresponds to zero stresses and zero resilience. As in Article 24, the quantities, $e_{na} \ldots\ldots e_{ni} \ldots\ldots$, etc., are called, by definition, "the eccentricities of the parameters with respect to the load, P." The stiffness constants, such as the coefficients in the resilience quadric, are assumed again to remain the same as in the original structure. When the parameters and the deflections are measured from the original shape of the structure, that is, when the set of values, $u_{na} = e_{na}, \ldots\ldots u_{ni} = e_{ni}$, etc., represents the initial shape of the changed eccentric structure, the expression

for the displacement, δ, in the direction of P will remain the same as in the original structure. According to Article 20, we have then Formula (87):

$$\delta = \Sigma\, \delta^{(n)} + \gamma\, v$$

where (Formula 89),

$$\delta^{(n)} = \frac{1}{2} \sum_i \delta_{ni}\, u_{ni}^{\,2} + \sum_{i,\,k} \delta_{ni,\,nk}\, u_{ni}\, u_{nk}$$

The expression for the resilience, U, is modified so as to make the set of values, $u_{na} = e_{na} \ldots\ldots u_{ni} = e_{ni} \ldots\ldots$, give $U = \text{min.}$, that is, instead of Formula (86), we have for the eccentric structure:

$$U = \Sigma\, \bar{U}^{(n)} + \frac{1}{2}\, V v^{\,2} + T\,(t_1, t_2, \ldots .) \ldots\ldots\ldots\ldots (120)$$

where,

$$\bar{U}^{(n)} = \frac{1}{2} \sum_i Q_n\, \delta_{ni}\, (u_{ni} - e_{ni})^2$$

$$+ \sum_{i,\,k} Q_n\, \delta_{ni,\,nk}\, (u_{ni} - e_{ni})\, (u_{nk} - e_{nk}) \ldots\ldots\ldots\ldots (121)$$

Substitution of Formulas (87), (89), (120), and (121) in the energy equation, $L = U - P\delta = \text{min.}$, and differentiation with respect to u_{ni} gives

$$\frac{\partial L}{\partial u_n} = \Big(Q_n - P\Big)\Big(\delta_{ni}\, u_{ni} + \sum_k \delta_{ni,\,nk}\, u_{nk}\Big)$$

$$- Q_n\, \Big(\delta_{ni}\, e_{ni} + \sum_k \delta_{ni,\,nkn} e_{nk}\Big) = 0 \ldots\ldots\ldots\ldots\ldots (122)$$

where $k \neq i$. By exchanging the indices, na, nb ....., for ni, a set of linear equations is obtained from which the parameters, u_{na}, u_{nb}, $u_{ni} \ldots\ldots$, belonging to Q_n can be determined. The solution of the set of Formula (122), is:

$$u_{na} = e_{na} + \frac{P}{Q_n - P}\, e_{na}, \;\ldots \qquad u_{ni} = e_{ni} + \frac{P}{Q_n - P}\, e_{ni}, \ldots\ldots (123)$$

Formula (123) is virtually the same as Formula (114), derived for the simplified case. Formula (123) should be compared with Formula (97), the general formula for astatic parameters in heterostatic action. The comparison also shows that, in the case of the eccentric action in the structure of more general character, the values of the parameters can be determined as the values generated in a certain heterostatic action in the original structure; and as, in the simplified case, this heterostatic action is produced by the load, P, which is an astatic load in the original structure, combined with an orthostatic load which would produce the eccentricities, $e_{na} \ldots\ldots e_{ni}$, etc.

As in the simplified case, one may measure the parameters starting from the initial shape of the eccentric structure, that is, instead of u_{ni}, one may use, $\bar{u}_{ni} = u_{ni} - e_{ni}$. This method gives, analogous to Formula (119).

$$u_{ni} = \frac{P}{Q - P}\, e_{ni} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (124)$$

With the quantities, $\bar{u}_{ni}$, introduced as parameters in the expressions for resilience and displacement, a form of the energy equation will be derived which is analogous to Formula (118). One finds, then, as in the simplified

case, that the eccentric action can be interpreted as a heterostatic action in the eccentric structure. In that action, both the orthostatic component and the astatic component of the load, are proportional to P, the astatic component differing from the load, P, on the original structure by a load equal and opposite to the orthostatic component.

If a permanent set causes $v = v_e$ to be the value of the orthostatic parameter at zero stresses, then v_e added to the value of v produced in the original structure, gives the value of v in the changed structure, as measured from the initial shape of the original structure. This conclusion is easily verified by an application of the energy equation. In the same manner, if a permanent set causes one of the independent parameters, say, t_1, to take the value, t'_1, at zero stresses, then, as long as P is the only load, $t_1 = t'_1$ will remain as the constant value of that parameter. In other words, permanent sets represented by changes of the orthostatic parameter and of the independent parameter can be treated independently, that is, the effects due to such permanent sets can be combined with the effects due to the eccentricities of the parameters, $u_{na} \ldots\ldots u_{nt} \ldots\ldots$, etc., by simple addition.

26.—Effects in Eccentric Action.—As a typical example consider the hinged-end column, with a length, l, and a constant stiffness factor, EI. Deflections from the originally straight center line can be expressed as follows:

$$y = \Sigma u_n \sin \frac{n\pi x}{l},$$

where x is the distance from one end. The factors, u_n, will be used as the astatic parameters. The critical loads are expressed by the Euler formula,

$$Q_1 = Q = \frac{\pi^2 EI}{l^2}; \quad Q_n = n^2 Q$$

The column, which had originally a straight center line, is now bent in such a way that permanent sets are left. The deflections, e, which remain when the load is released and the stresses are zero, may be expressed by a Fourier series of the form:

$$e = \Sigma e_n \sin \frac{n\pi x}{l}$$

According to the definitions made in Articles 24 and 25, $e_1 \ldots\ldots e_n \ldots\ldots$ are then the eccentricities of the astatic parameters, $u_1 \ldots\ldots u_n \ldots\ldots$, with respect to the end load, P. Denote, further: c = the distance from the center line to the extreme fibers, so that $\frac{I}{c}$ = the section modulus; A = the area of cross-section; and k = the radius of gyration, defined by $I = Ak^2$. The stresses, s, in the extreme fibers are among the effects which have particular interest, if y is measured from the original straight center line, which is also the line of the force, P; in that case the stress is expressed as:

$$S = \frac{P}{A} + \frac{P\,yc}{I}; \text{ or, } S = \frac{P}{A}\left(1 + \frac{y\,c}{k^2}\right)$$

According to Formula (123) or Formula (114), we have:

$$u_n = e_n + \frac{P}{Q_n - P} e_n = e_n + \frac{P}{n^2 Q - P} e_n$$

By substituting these values in the expression for y, and then introducing the value of y in the expression for S, we find:

$$S = \frac{P}{A}\left(1 + \frac{c}{k^2} (e + \Sigma \frac{P}{n^2 Q - P} e_n \sin \frac{n\pi x}{l}\right) \ldots\ldots\ldots (125)$$

where $n = 1, 2, 3 \ldots\ldots$

Formula (125) gives the stress in the extreme fibers at any one point of the column. The reversed problem, that of finding the limiting value of P corresponding to a maximum allowable stress, S, may be treated by solving Formula (125), with respect to P. P is then found as a function of x and l. For each given value of l, the distance, x, has to be chosen so as to make P a minimum. In this manner, the limiting values of P will be found as a function of l. This function, say, $P = P(l)$, can be considered as the solution of Formula (125). In actual columns, it is impossible to secure an absolutely straight center line, an absolutely concentric loading, or a perfect evenness of the elastic qualities. The result is that eccentricities, such as those measured by e, or by $e_1, \ldots\ldots e_n$, cannot be avoided altogether in actual columns. With definite, properly chosen values of $e_1 \ldots\ldots e_n \ldots\ldots$, Formula (125), therefore, can be considered as a general column formula applying to actual columns, as long as their action depends essentially on the conditions before the elastic limit has been exceeded.* The column formulas commonly used in engineering design, such as that of Rankine, or the combination of the Euler formula with Johnson's parabolic formula, therefore, should appear as approximations to solutions of Formula (125). In the analysis of columns, one sometimes assumes a constant eccentricity which should then be considered as an "equivalent constant eccentricity", producing the same effect as the actual varying eccentricity. With $e =$ constant, Formula (125) would lead to a solution, but the case is solved more easily by direct application of the differential equation of flexure. The result is the well known secant formula, which is given in many textbooks on Strength of Materials. With other end conditions, fixed ends, one free end, etc., formulas quite similar to Formula (125) can be derived. It may be said then that Formula (125) is a column formula of a general type, which applies to actual columns having small definite eccentricities.

If in the case solved by Formula (125), the greatest stress can be expected to be at or near the middle of the column, the solution can be simplified by introducing the definite value, $x = \frac{l}{2}$. This value gives $\sin \frac{n\pi x}{l}$ equal to: zero when n is even; $+1$ when $n = 1, 5, 9, \ldots$; and -1 when $n = 3, 7, \ldots\ldots$ Thereby, Formula (125) is reduced to:

$$S = \frac{P}{A}\left(1 + \frac{c}{k^2}\left(e_{center} + \frac{P}{Q - P} e_1 - \frac{P}{9Q - P} e_3 + \frac{P}{25Q - P} e_5 \ldots\right)\right). (126)$$

where $e_{center} = e_1 - e_3 + e_5 \ldots\ldots$

* Compare Kármán's work in which the influence of stresses above the elastic limit is investigated, "Mitteilungen über Forschungsarbeiten auf dem Gebiete des Ingenieurwesens", v. 81 (1910).

The general case of eccentric action in any structure may now be considered. It is assumed that the load, P, would produce pure astatic action if it was not for the eccentricities, e_{na}, e_{nb}......e_{ni}......, etc., of the astatic parameters, u_{na}, u_{nb}......u_{ni}......, etc. One may consider some "effect", F. F will be assumed to be an effect which is expressed by the same function of the parameters whether the original or the eccentric structure is considered, the parameters being measured in both cases, from the initial shape of the concentric structure. In the special case previously treated, the stress, S, when expressed as indicated in terms of the deflections, y, is an effect of this kind, and so is, in general, any effect which is proportional to the deflections from the original shape. The general effect, F, was expressed by Formula (99), which, after substitution of Formula (100), may be written in the form:

$$F = \sum_n \sum_i F_{ni}\, u_{ni} + F_0\, v + \overline{F}\,(t_1, t_2, \ldots\ldots) \ldots\ldots\ldots\ldots(127)$$

When P acts alone on the original concentric structure, and is different from any of the critical values—so that no buckling takes place — the effect is proportional to the orthostatic parameter, and may be written, in accordance with Formulas (127) and (83), as follows:

$$F_c = F_0\, v = P\,\frac{F_0\,\gamma}{V}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(128)$$

Let t_1', t_2'......, be the eccentricities caused in the independent parameters, t_1, t_2......, by the permanent set. By substituting these values, Formula (128), and the values of the parameters, u_{ni}, given by Formula (123), in Formula (127), the effect in the eccentric action, is found to be as follows:

$$F = F_c + \sum_n \sum_i F_{ni}\, e_{ni} + \sum_n \frac{P}{Q_n - P} \sum_i F_{ni}\, e_{ni} \; + \overline{F}\,(t_1', t_2', \ldots). \;(129)$$

Formula (129), as might be expected, is quite similar to Formula (101), which determines the effects, F', in heterostatic action. As a matter of fact, Formula (129) might have been derived as a special case of Formula (101).

If F is an effect which varies from point to point, then F and the coefficients, F_{ni}, are functions of the location. Assume that the limiting strength of the structure is defined by some limiting value of F, which must not be exceeded. The lowest value of P found by solving Formula (129), with this value of F substituted, determines the upper limit beyond which the load, P, cannot be increased. In other words, Formula (129), like Formula (125) in the special case, can be used as a general formula to define the carrying capacity in eccentric action. It is practically impossible to build any structure absolutely exactly as designed. It follows that small eccentricities such as the e's in Formulas (125) and (129), must always be expected to exist. Equations such as Formula (129), with certain assigned values of the eccentricities, therefore, may be used to determine the limiting values of loads which are apparently astatic loads, and would be so if the structure was built with absolute exactness. It follows from the similarity between Formulas (129) and (125) that in a great number of the cases covered by the general Formula (129), it will be possible to substitute approximate formulas similar to the

solutions of Formula (125); that is, formulas similar to that of Rankine, Johnson's parabolic formula, etc., will be approximate solutions of the general Formula (129) in various cases of buckling.

27.—Determination of Astatic Actions and Astatic Parameters by the Expressions for Resilience and Displacement.—Assume that a certain structure is defined with sufficient exactness by a finite number of parameters, x_1, x_2......, which are not assumed, in general, to be astatic parameters. Let the resilience be expressed by the quadric:

$$U = \frac{1}{2} A_1 x_1^2 + \frac{1}{2} A_2 x_2^2 + \ldots + A_{1,2} x_1 x_2 + \ldots \qquad (130)$$

Assume that the displacement in the direction of the load, P, is also expressed by a quadric, namely,

$$\delta = \frac{1}{2} a_1 x_1^2 + \frac{1}{2} a_2 x_2^2 + \ldots + a_{1,2} x_1 x_2 + \ldots \qquad (131)$$

Then, the energy equation, $L = U - P\delta = \text{min.}$, gives:

$$\left.\begin{array}{l} \dfrac{\partial L}{\partial x_1} = (A_1 - a_1 P) x_1 + (A_{1,2} - a_{1,2} P) x_2 \\ \qquad + (A_{1,3} - a_{1,3} P) x_3 + \ldots = 0 \\ \dfrac{\partial L}{\partial x_2} = (A_{1,2} - a_{1,2} P) x_1 + (A_2 - a_2 P) x_2 \\ \qquad + (A_{2,3} - a_{2,3} P) x_3 + \ldots = 0 \\ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \end{array}\right\} \qquad (132)$$

The equations, Formula (132), are solved by $x_1 = x_2 = \ldots\ldots = 0$. Solutions different from zero are possible, however, when the determinant of the equations, Formula (132), vanishes,* that is, when:

$$\left.\begin{array}{lll} A_1 - a_1 P & A_{1,2} - a_{1,2} P & A_{1,3} - a_{1,3} P \ldots \\ A_{1,2} - a_{1,2} P & A_2 - a_2 P & A_{2,3} - A_{2,3} P \ldots \\ \ldots\ldots & \ldots\ldots & \ldots\ldots \end{array}\right\} = 0 \ldots \quad (133)$$

If there are m parameters, Formula (133) is an equation of mth degree in P, and has, in general, m solutions. Assume that these solutions are $P = Q_1$, Q_2..... Q_n......, and take, for instance, $P = Q_n$. Substituted in Formulas (132), $P = Q_n$ gives the set:

$$\left.\begin{array}{l} (A_1 - a_1 Q_n) x_1 + (A_{1,2} - a_{1,2} Q_n) x_2 + (A_{1,3} - a_{1,3} Q_n) x_3 + \ldots = 0 \\ (A_{1,2} - a_{1,2} Q_n) x_1 + (A_2 - a_2 Q_n) x_2 + (A_{2,3} - a_{2,3} Q_n) x_3 + \ldots = 0 \\ \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \end{array}\right\} . (134)$$

With $x_1 = c_{n_1} u_n$, where c_{n_1} is a chosen constant, and u_n = any value, $m - 1$, of the equations, Formula (134), gives solutions which satisfy the remaining equation, hence, the solutions of the equations, Formula (134), may be expressed, as follows:

$$x_1 = c_{n1} u_n\,; \qquad x_2 = c_{n2} u_n\,; \qquad x_3 = c_{n3} u_n\,; \ldots\ldots \quad (135)$$

where u_n can take any value. u_n can be used as a parameter defining the variation expressed in Formula (135). Furthermore, u_n can be interpreted as

* The vanishing of a determinant is used by several writers as a criterion of buckling or of an equilibrium ceasing to be stable; for example, R. V. Southwell, *Philosophical Transactions*, Royal Soc., A, v. 213 (1914), p. 218; A. Ostenfeld, "Teknisk Statik", II, ed. 1913, p. 490; H. Zimmerman, Sitzungsber. der k. preuss. Akad. der Wissenschaften (1909), p. 193.

an astatic parameter. Its corresponding critical load is $P = Q_n$. By making $P = Q_1, Q_2 \ldots\ldots$, successively, and repeating the same operations, the set of astatic parameters, $u_1, u_2 \ldots\ldots$, may be determined completely.

One conclusion reached is that pure astatic action occurs when both the displacement in the direction of the load and the resilience are quadrics in the parameters. It is easy to verify the fact that this conclusion will continue to hold when additional terms, $\frac{1}{2} V v^2 + T(t_1, t_2 \ldots)$, in U, and γv in δ, are introduced. v and $t_1, t_2 \ldots$ will be recognized at once as orthostatic and independent parameters, respectively.

If δ had been a linear function of the parameters instead of a quadric, P would have been an orthostatic load. In a case of more general nature, the displacement in the direction of the load, P, is a quadratic function containing some linear terms. The total displacement may then be expressed as $\delta' = \delta + w$, where δ is a quadric of the form, Formula (131), while w contains the linear terms. In this case, the total load, P, may be interpreted as consisting of two components, each measured in intensity by the quantity, P. One component of P is an astatic load in the direction of the displacement, δ, the other component of P is an orthostatic load in the direction of the displacement, w. One concludes that if the resilience is a quadric in terms of a certain set of parameters, and if the displacement in the direction of a given load is expressed in terms of these parameters as a quadratic function containing linear terms, the action produced is a heterostatic action. This heterostatic action is recognized as one of the general type produced under the combined influence of an orthostatic and an astatic component of the load.

V.—Differential Equations Applying to Some Definite Cases.

28.—The Methods of Solution: Differential Equations and the Energy Method.—The problems treated herein are usually solved by one of two fundamental methods, one of which is based on the energy principle, and the other on the direct statical principles of equilibrium. Both methods are represented at many places in the works mentioned in the Bibliography (Part VII). The general theory in Part IV of this investigation was based on the energy principle. The problem of equilibrium is reduced by the energy principle to a problem of energy minimum, or, to a variation problem. The analyses in Part IV show clearly that this method of attack is particularly effective in the derivation of general principles; but the history of technical statics has shown that the principle of least action can be applied with advantage also to structural problems of definite and specific character. The fundamental work of Castigliano* on statically indeterminate structures is an example. Castigliano's method of least work was applied primarily to problems the solution of which depends on a finite number of unknown or "indeterminate" quantities. In 1909, W. Ritz† indicated a method by which Castigliano's principles can be

* A. Castigliano, "Théorie de l'équilibre des systèmes élastiques et ses applications", Torino, 1879.

† W. Ritz, "Ueber eine neue Methode zur Lösung gewisser Variationsprobleme der mathematischen Physik", Crelles' *Journal für reine und angewandte Mathematik*, v. 135 (1909).

used even when the structure depends on an infinite number of "indeterminate" parameters. Results are then expressed in series giving successive approximations. Ritz's method has been applied since by several writers in the analysis of continuous structures, such as slabs, domes, water tanks, etc. S. Timoshenko* used the method extensively in the analysis of astatic action. The case of a column with varying section, for example, is analyzed conveniently by Ritz's method.† The conclusion is that the energy principle can be used with advantage in the analysis of the general questions and of some of the more complex specific questions.

Other cases are analyzed, however, as easily or more easily by the use of the direct statical principles of equilibrium. These principles are expressed in most cases of astatic or heterostatic action by certain differential equations which represent the conditions of equilibrium of each element of the structure. Thus, the solution of the problem is made to depend on the solution of certain differential equations. This method is advantageous in the analysis of the simple structures, such as single columns, curved members, and slabs. In the following Articles, the derivation of the differential equations of heterostatic action in such structures will be indicated. By omitting either the astatic or the orthostatic component of the load, one may analyze the orthostatic or the astatic actions, respectively, and, in particular, one may determine the nature of the astatic parameters. Afterward, the heterostatic action may be investigated by the formulas for combination which were indicated in Part IV, particularly by Formulas (74), (75), (97), (101), and (102). The investigation will not, except in special cases, proceed further than the determination of these differential equations. As to their solution in detail, and their application to individual cases, reference is made to the several specific treatments which are listed in the Bibliography (Part VII).

29.—Straight Columns.‡—Cases in which the axial load is applied continuously throughout the length of the column, or in which there is a continuous transverse elastic support, have been analyzed by Timoshenko, Greenhill, and Zimmerman.§ Such columns will not be included in the present analysis. The discussion will be confined to cases in which the axial load is transferred at a finite number of definite points and in which the transverse support is at definite points. In such cases, the column may be divided into sectors, each of which is between two consecutive points at which there is a transverse reaction or a change of the axial load. Fig. 9 shows such a sector. The distances, x, along the column are measured from the lower end of this sector. The x-axis may be fixed in space or attached in some way to points of the center line of the column. The deflections, y, are measured from the x-axis, positive toward the right. The axial load, P, is the astatic component of the load. P may be inclined relative to the x-axis as shown in Fig. 9 (a), but, in this case, it may always be replaced, as indicated in Fig. 9 (b), by a load, P, along the x-axis combined with a horizontal force, X, and a couple, Y, at the lower end.

* See Bibliography, Part VII, Section *A*.

† See the works by Timoshenko previously mentioned, or, A. Ostenfeld, "Teknisk Elasticitetslære", 3d Edition (1916), p. 442.

‡ Bibliography, Part VII, Section *C*.

§ Bibliography, Part VII, Sections *A* and *C*.

X and Y may be considered as constants which are to be determined by the end conditions. Their positive directions are shown in Fig. 9 (*b*). The remainder of the notation follows:

Let M = bending moment due to the orthostatic or transverse load alone. M is considered as positive when causing compression to the left, tension to the right.

M' = bending moment under the combined influence of the transverse and the axial load, considered positive in the same direction as M.

EI = stiffness factor (modulus of elasticity times moment of inertia of the cross-section). EI may vary as a function of x.

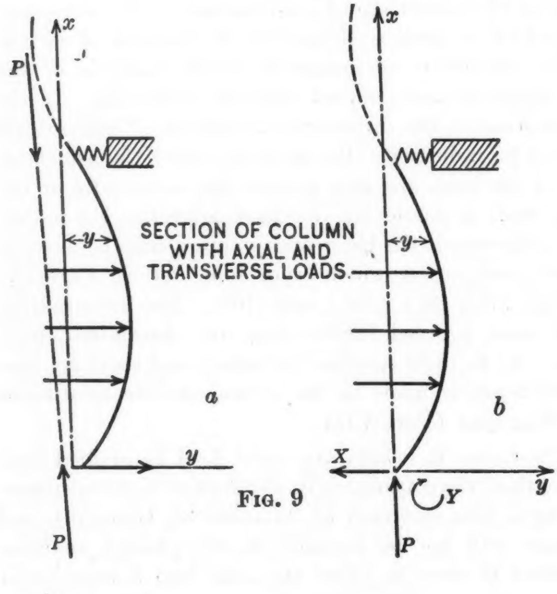
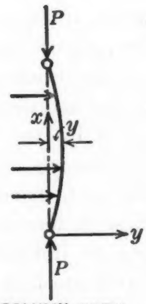
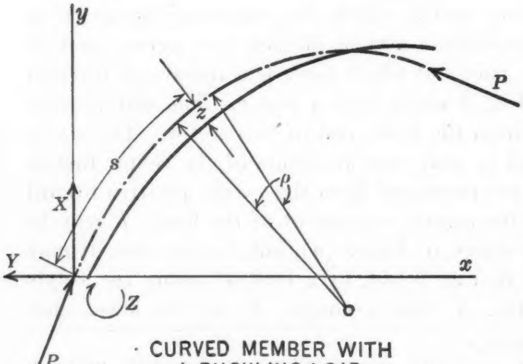


FIG. 9



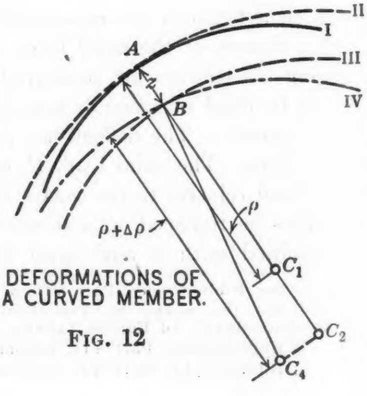
COLUMN WITH TRANSVERSE LOADS

FIG. 10



CURVED MEMBER WITH A BUCKLING LOAD.

FIG. 11



DEFORMATIONS OF A CURVED MEMBER.

FIG. 12

With positive directions as indicated, the equation of flexure is,

$$E\,I\,\frac{d^2\,y}{d\,x^2} = -\,M' \qquad (136)$$

From Fig. 9 (*b*), it follows that:

$$M' = M + Py + Xx + Y \qquad (137)$$

therefore, the differential equation applying to the sector is:

$$E\,I\,\frac{d^2\,y}{d\,x^2} + Py + Xx + Y = -\,M \qquad (138)$$

where the constants, X and Y, are to be chosen so as to satisfy the end conditions. The end conditions are defined by the character of the supports, by the methods of transfer of the axial load, or by the connection with adjoining sectors above and below.

$P = X = Y = 0$ gives orthostatic action, which may be analyzed by the common methods of technical mechanics. $M = 0$, $EI =$ constant, P positive (compression) gives solutions of the type:

$$y = C_1 \sin x \sqrt{\frac{P}{E\,I}} + C_2 \cos x \sqrt{\frac{P}{E\,I}} - \frac{Xx + Y}{P} \qquad (139)$$

where C_1 and C_2 as X and Y are constants. $M = 0$, $EI =$ constant, P negative (representing the case of tension) gives solutions of a form similar to Formula (139), but with hyperbolic sine and cosine instead of the sine and the cosine. When both ends of the sector are hinged, and the load is applied, as in Fig. 10, then these particular end conditions give $X = Y = 0$ in Formula (138). When $M = 0$, $EI =$ constant, the solution, Formula (139), applies, but is in this case reduced to the well-known form:

$$y = C_1 \sin x \sqrt{\frac{P}{E\,I}} \qquad (140)$$

Formulas (139) and (140) define the particular astatic actions dealt with in Part II. Formula (139) applies also to such cases of eccentric action, or action with end couples, as were treated in Article 9. The formula leads directly to Formula (15) for the bending moment produced in these actions.

*30.—Curved Members.**—Fig. 11 shows a curved elastic member. The dotted curve indicates the undeflected center line, and the full curve the deflected center line. Both curves are assumed to be contained in the xy-plane. The origin is at one end of the undeflected curve. We shall begin by considering the special case in which astatic action is produced by a load system, P, consisting of the following forces: First, two equal forces, P, tangent to the undeflected center line, are acting at the ends toward the structure so as to produce mainly compression. Second, a distributed normal pressure, p, is applied on the convex side of the curve; the distribution of p is such that the undeflected center line is a funicular curve (line of pressure) for those pressures; the intensity of the pressures, p, varies in such a way proportional to the end forces, P, that the end forces hold the distributed pressure in equilibrium. Third, at the lower end (at the origin), there is, in addition to

* See Bibliography, Part VII, Section G.

the concentrated force, P, an end load which may be resolved into the three components X, Y, and Z. The two forces, X and Y, act through the origin in the directions $+y$ and $-x$, respectively (as indicated by the arrows in the diagram); and the couple, Z, acts in a clockwise direction. X, Y, and Z may be considered as reactions produced by the remainder of the total astatic load, P, and are zero when $P = 0$. Fourth, at the upper end there is, in addition to the concentrated force, P, a load similar to that of the load, X, Y, Z, at the lower end. All these loads are parts of the astatic component, P, of the total load. In the general case of heterostatic action, there is, in addition, an orthostatic load-component, W.

It is further denoted that:

s = the distance measured along the curve from the origin; and

z = the deflection normal to the curve, positive toward the right when the observer looks in the direction of increasing s.

ρ = the radius of curvature, measured positive from the curve in the positive direction of z.

M, M', and EI = the same as in Article 29. The moments are considered as positive when they cause compressions to the left when the observer looks in the direction of increasing s.

The orthostatic load may produce not only bending moments, M, but also axial pressures. The latter will be assumed to be small compared with the astatic load, P. Then, it may be assumed, with a sufficient approximation as far as the stresses and deflections are concerned, that the axial pressure in the heterostatic action is the same as in the astatic action produced by P. This procedure is equivalent to considering the orthostatic load as replaced by an equivalent load which produces the same bending moment, M, but no axial pressures.

It is assumed that the center line is not a very sharp curve, so that the radius of curvature at any point is several times the transverse dimensions of the structural member. Then the equation of flexure may be written:

$$\frac{1}{\rho + \Delta\rho} - \frac{1}{\rho} = -\frac{M'}{EI} \qquad \text{(141)}$$

The left-hand expression may be transcribed as follows:* In Fig. 12, the Curve I, is an element of the undeflected curve, while Curve IV represents the same curve element in the deflected position. The transformation of the element, Curve I, into the element, Curve IV, may take place through the intermediate positions, Curves II and III, which are defined as follows: Curve II is tangent to Curve I at the point A, which is under consideration; Curve III has the same center of curvature, C_2, as Curve II, and the same radius of curvature as Curve IV. The centers of curvature of the four elements in Fig. 12 are C_1, C_2, C_z, and C_4, respectively. Let z_2 denote the deflection of Curve II relative to Curve I, then the increase of curvature

* Another derivation based on the formula for the radius of curvature in polar co-ordinates is given in A. Ostenfeld's "Teknisk Elasticitetslære", 3d ed. (1916), p. 480.

(curvature = reciprocal of radius of curvature) at the transformation, Curve I to Curve II, is expressed as $\frac{d^2 z_2}{ds^2}$. At the transformation, Curve II to Curve III, the increase of curvature is $\frac{1}{BC_2} - \frac{1}{AC_2}$, but when C_1, C_2, and the deflection $AB = z$, are small distances, this difference is very nearly equal to, and may be replaced by, the quantity $\frac{1}{BC_1} - \frac{1}{AC_1} = \frac{z}{\rho^2}$. The transformation, Curve III to curve IV, does not change the curvature any further. Since $\frac{d^2 z}{ds^2} = \frac{d^2 z_2}{ds^2}$ it follows that the total increase of curvature, or the left side in Formula (141), is $\frac{d^2 z}{ds^2} + \frac{z}{\rho^2}$. Hence, the equation of flexure can be written as follows:

$$EI\left(\frac{d^2 z}{ds^2} + \frac{z}{\rho^2}\right) = -M' \qquad (142)$$

Consideration of Fig. 11 gives in the heterostatic action,

$$M' = M + Pz + Xx + Yy + Z,$$

and this expression, substituted in Formula (142), gives the equation of flexure:

$$EI\left(\frac{d^2 z}{ds^2} + \frac{z}{\rho^2}\right) + Pz + Xx + Yy + Z = -M' \qquad (143)$$

The corresponding Formula (138), applying to straight columns, represents the special case of Formula (143), in which $\rho = \infty$.

With ρ = constant, EI = constant, $M = 0$, and with certain end conditions, the solution of Formula (143) is a trigonometric function, sine or cosine of angles proportional to s. Such solutions define the astatic actions of circular cylinders carrying a uniform outside pressure.

In deriving the differential equation, Formula (143), it was assumed that the pressure, p, was at all places perpendicular to the curve s. Under such circumstances the axial pressure in the astatic action is the same in all cross-sections, and equal to the end pressure, P. However, an astatic action is also possible, in which the axial pressure, N, varies. The variation from the value, P, which is the value of N at the point, $s = 0$, is caused by tangential components of the distributed pressure, p. The general condition of pure astatic action is that the undeflected center line is a pressure line for the force set, P, p. The axial forces, N, appear as rays in the corresponding force polygon (or force curve). It follows that N at each point varies proportionally to P, so that P may still be used as a measure of the intensity of the astatic load. When the force set, P, p, can be assumed to remain in a constant position after the flexure, then the differential equation will be the same as Formula (143), except that N is substituted for P. As in the special case, the constants, X, Y, and Z, as well as the integration constants must be determined from the end conditions.

*31.—Slabs.**—Fig. 13 shows a part of the slab in its undeflected position. The xy-plane is the central plane of the undeflected slab. The y-axis is assumed to be vertical, and the x-axis to be horizontal. The astatic load consists of the following uniformly distributed forces, all parallel to the xy-plane, and acting in the xy-plane as long as the slab is undeflected:

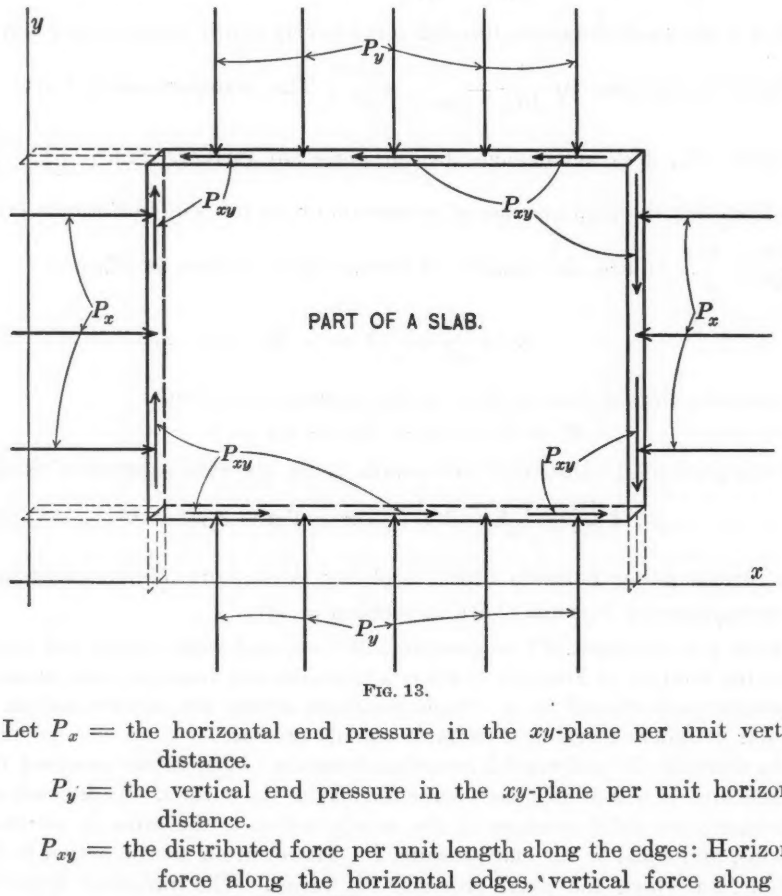


FIG. 13.

Let P_x = the horizontal end pressure in the xy-plane per unit vertical distance.

P_y = the vertical end pressure in the xy-plane per unit horizontal distance.

P_{xy} = the distributed force per unit length along the edges: Horizontal force along the horizontal edges, vertical force along the vertical edges. These horizontal and vertical forces are equal per unit length on account of the law of "shearing forces in pairs".

The orthostatic load is:

w = the surface pressure perpendicular to the slab per unit area. In the general case, w is a function of x and y.

* See Bibliography, Part VII, Section *J*. The derivation of the equation of flexure of slabs in rectangular co-ordinates with loads normal to the surface is given in several treatises, see, for example, A. Föppl, "Technische Mechanik", v. 5, 1907 ed., pp. 97 *et seq.* Concerning the terms in the flexure equation, which are due to the end pressures see, for example, A. E. H. Love, "Theory of Elasticity" (1906), pp. 528-529. A flexure equation of general form, which includes the effects of end pressures and end shears, is given in B. de Saint-Venant's annotated translation of Clebsch's "Theory of Elasticity", Paris, 1883, Note by Saint-Venant, p. 704, Equation (e_1).

The pressures, P_x and P_y, give uniform horizontal and vertical compressions in the undeflected slab, while P_{xy} produces a uniform horizontal and vertical shear. It is assumed that this uniform distribution will still exist after the slab has deflected, and this assumption is generally warranted as long as the deflections are small compared with the thickness of the slab. The conclusion is that P_x, P_y, and P_{xy} will be transmitted as constant internal pressures and shears parallel to the xy-plane. They may be interpreted as being located centrally in the slab, only they must then be considered as combined with other internal forces, namely, such as represent the usual bending stresses.

Fig. 14 shows the forces acting on the elements, $dxdy$, after it has deflected out of its original position. The deflections are defined as follows:

z = the deflection of lower left-hand corner, Point x, y;

$\frac{\partial z}{\partial x} dx$ = increase of deflection from left face to right face; and

$\frac{\partial z}{\partial y} dy$ = increase of deflection from lower face to upper face.

The loads shown acting on the element in Fig. 14 may be listed as follows: The surface load, $w\ dxdy$, in the direction, z; the central internal forces parallel to the xy-plane, namely, $P_x dy$ and $P_{xy} dx$, in the x-direction, and $P_y dx$ and $P_{xy} dy$ in the y-direction; in addition, internal shears in the directions, $\pm z$, and bending moments and torsional moments, as given in Table 4. The unit torsional moments, Z, in the horizontal and vertical faces are equal on account of the law of equality of shears in sections perpendicular to one another.

TABLE 4.—Forces and Couples Acting on the Slab Element in Fig. 14.

	Shear in directions, $\pm z$	Bending moments	Torsional moments
Left face	$V_x\ dy$	$X\ dy$	$Z\ dy$
(Direction)	$(-z)$	(xz)	(yz)
Right face	$\left(V_x + \frac{\partial V_x}{\partial x} dx\right) dy$	$\left(X + \frac{\partial X}{\partial x} dx\right) dy$	$\left(Z + \frac{\partial Z}{\partial x} dx\right) dy$
(Direction)	$(+z)$	(zx)	(zy)
Lower face	$V_y\ dx$	$Y\ dx$	$Z\ dx$
(Direction)	$(-z)$	(yz)	(xz)
Upper face	$\left(V_y + \frac{\partial V_y}{\partial y}\right) dx$	$\left(Y + \frac{\partial Y}{\partial y} dy\right) dx$	$\left(Z + \frac{\partial Z}{\partial y} dy\right) dx$
(Direction)	$(+z)$	(zy)	(zx)

By equating the sum of the forces in the z-direction to zero, and dividing by $dx\ dy$, the following condition is found:

$$\frac{\partial V_x}{\partial x} + \frac{\partial V_y}{\partial y} + w = 0 \quad \text{.................. (144)}$$

By equating the sum of the moments about a vertical line through the center of the element to zero, and dividing by $dx\,dy$, we find:

$$\frac{\partial X}{\partial x} + \frac{\partial Z}{\partial y} = V_x + P_x \frac{\partial z}{\partial x} + P_{xy} \frac{\partial z}{\partial y} \ldots\ldots\ldots\ldots (145)$$

In the same way, by using a moment axis through the center, parallel to the x-axis, we find:

$$\frac{\partial Y}{\partial y} + \frac{\partial Z}{\partial x} = V_y + P_y \frac{\partial z}{\partial y} + P_{xy} \frac{\partial z}{\partial x} \ldots\ldots\ldots (146)$$

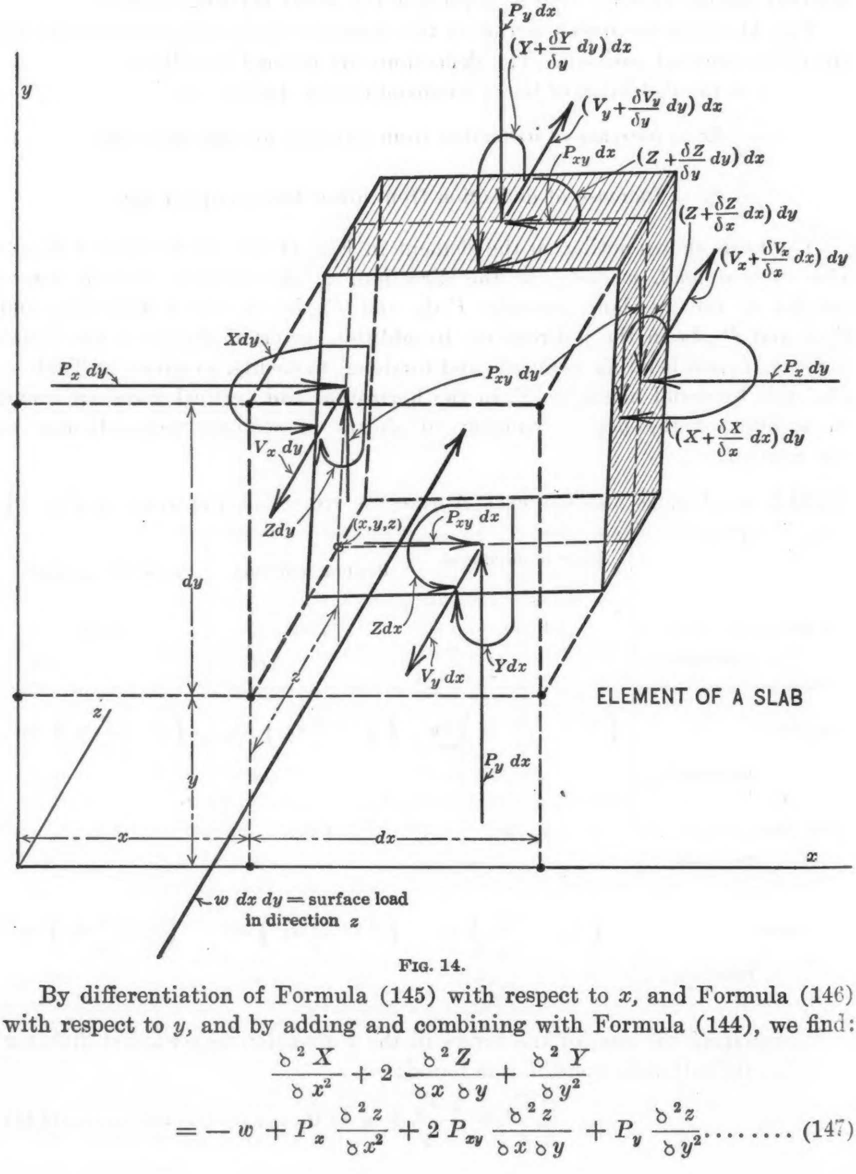


FIG. 14.

By differentiation of Formula (145) with respect to x, and Formula (146) with respect to y, and by adding and combining with Formula (144), we find:

$$\frac{\partial^2 X}{\partial x^2} + 2\,\frac{\partial^2 Z}{\partial x\,\partial y} + \frac{\partial^2 Y}{\partial y^2}$$

$$= -\,w + P_x \frac{\partial^2 z}{\partial x^2} + 2\,P_{xy} \frac{\partial^2 z}{\partial x\,\partial y} + P_y \frac{\partial^2 z}{\partial y^2} \ldots\ldots\ldots (147)$$

Formula (147) is a general equation of equilibrium on which theory dealing with specific cases may be based. It applies to continuous slabs whether or not they are homogeneous, and whether they are isotropic, that is, having the same properties in different directions, or æolotropic, having different properties in different directions. An example of the latter is the two-way, reinforced concrete slab, in which one may have to assume different stiffnesses in the various directions. Formula (147) applies also to the case of the double system of closely spaced beams for which buckling formulas were given in Article 11. These formulas may be derived from Formula (147) by putting $Z = 0$, as corresponding to the condition of zero torsional resistance, by putting $P_{xy} = 0$ and $w = 0$, and by substituting terms for X and Y, which are proportional to the curvatures in the two directions. Then, values of P_x and P_y, for which solutions different from zero exist, are critical values. When the x- and y-axes are along two edges, such solutions have the following form:

$$z = u_{mn} \sin \frac{m \pi x}{a} \sin \frac{m \pi y}{b} \dots\dots\dots\dots\dots\dots (148)$$

where u_{mn} is a constant which can be used as an astatic parameter, while m and n are integers.

A particularly important case is that of a homogeneous slab, for which buckling formulas were indicated in Article 10.

Assume that EI = the modulus of elasticity times the moment of inertia of cross-section per unit length; and

K = Poisson's ratio of lateral contraction to longitudinal elongation.

Then, the usual slab theory (see, for instance, the previously quoted treatment in Föppl's work) gives the following relation between deformations and moments in the homogeneous slab:

$$\frac{\partial^4 z}{\partial x^4} + 2 \frac{\partial^4 z}{\partial x^2 \partial y^2} + \frac{\partial^4 z}{\partial y^4}$$
$$= - \frac{1 - K^2}{E\,I} \left(\frac{\partial^2 X}{\partial x^2} + 2 \frac{\partial^2 Z}{\partial x \partial y} + \frac{\partial^2 Y}{\partial y^2} \right) \dots\dots (149)$$

Combination with Formula (147) gives then the formula for heterostatic action in homogeneous slabs, namely:

$$\frac{E\,I}{1 - K^2} \left(\frac{\partial^4 z}{\partial x^4} + 2 \frac{\partial^4 z}{\partial x^2 \partial y^2} + \frac{\partial^4 z}{\partial y^4} \right) + P_x \frac{\partial^2 z}{\partial x^2}$$
$$+ 2 P_{xy} \frac{\partial^2 z}{\partial x \partial y} + P_y \frac{\partial^2 z}{\partial y^2} = w \dots\dots\dots\dots\dots (150)$$

As an example, consider the rectangular slab treated in Article 10, which is simply supported along straight edges, and has the spans, a and b. It is assumed that no shearing forces like P_{xy} are acting. With $w = 0$, $P_{xy} = 0$, and with co-ordinate axes along two edges, Formula (150) will be satisfied by solutions of the form, Formula (148), on the condition that P_x and P_y take such special, mutually dependent values, Q_a and Q_b, as are indicated by Formula (21) in Article 10. These are the critical values which produce astatic actions.

The writer will indicate how the bending moment at the center of a simply supported rectangular slab may be found by integration of Formula (150). He will first consider the orthostatic action produced by a uniform surface load, w, acting alone. The computations are simplified by assuming that:

$$b = \pi \quad = \text{short span.}$$

$$a = \frac{\pi}{\alpha} = \text{long span.}$$

$$w = \frac{1}{\pi^2} = \text{surface load per unit area.}$$

These values give $wb^2 = 1$. The use of a as the ratio of the spans is in accordance with the notation in Article 10, and the results found with these special values may be made to apply in the more general case by multiplication by the proper factors. For example, wb^2, with the notation used in Article 10, is the factor used in expressing the bending moments per unit width.

The x-axis and the y-axis are taken parallel to the long and to the short span, respectively. The origin is at the center, and the surface load, w, may be expressed in a double Fourier series, as follows:

$$w = \frac{16}{\pi^4} \sum_{1,3..}^{m} \sum_{1,3..}^{n} \frac{-(-1)^{\frac{m+n}{2}}}{m\ n} \cos m\,\alpha\,x \,.\, \sin n\,y \;=\; \frac{1}{\pi^2} \ldots (151)$$

This expression is substituted in Formula (150). In the orthostatic action, we have $P_x = P_y = P_{xy} = 0$. Then, we find the following solution of Formula (150):

$$z = \frac{16}{\pi^4} \frac{(1 - K^2)}{E\,I} \sum_{1,3..}^{m} \sum_{1,3..}^{n} \frac{-(-1)^{\frac{m+n}{2}}}{m\,n\,(\alpha^2\,m^2 + n^2)^2} \cos m\,\alpha\,x \,.\, \cos n\,y \,..(152)$$

In indicating formulas for bending moments in orthostatic action in Article 10, Poisson's ratio K, was assumed to be equal to zero. Reasons for deriving formulas on this basis, even when K is known to differ from zero, were stated in Article 10. With $K = 0$, the bending moments are directly proportional to the curvatures. The following expression is then found for the bending moment in the short span:

$$Y = -E\,I \frac{\delta^2 z}{\delta\,y^2}$$

$$= \frac{16}{\pi^4} \sum_{1,3..}^{m} \sum_{1,3..}^{n} \frac{-(-1)^{\frac{m+n}{2}}\,n}{m\,(\alpha^2\,m^2 + n^2)^2} \cos m\,\alpha\,x \,.\, \cos n\,y \ldots\ldots (153)$$

By introducing the factor, wb^2, on the right side, Formula (153) can be made to apply to any spans and unit load. Then, by substituting $x = y = 0$, which corresponds to the center of the slab, the following expression is found for the bending moment in the short span at the center:

$$M_b = \frac{16\,w\,b^2}{\pi^4} \sum_{1,3..}^{m} \sum_{1,3..}^{n} \frac{-(-1)^{\frac{m+n}{2}}\,n}{m\,(\alpha^2\,m^2 + n^2)^2} \ldots\ldots\ldots (154)$$

This double infinite series defines a function of wb^2 and a. It can be shown that the function, Formula (154), is rather well approximated by the simplified Formula (28), which was indicated in Article 10.

Each separate term in the double infinite series, Formulas (152), (153), and (154), is seen to represent a case of pure astatic equilibrium as produced by the end loads, P_x and P_y, alone (compare Formula (148)), that is, these series are of the type represented in Formula (99) which may be written in this case, as follows:

$$F = \Sigma F^{(n)} \ldots\ldots\ldots\ldots\ldots\ldots (155)$$

where F and $F^{(n)}$ are effects in orthostatic and astatic actions, respectively. Formula (155) may also be interpreted as a form of equation, Formula (60), which is the corresponding formula applying in the simplified case which was treated in Article 17. Formula (155) is also of the same form as Formula (11) in Article 7, which expresses the moments in columns. In accordance with Formula (102), the corresponding effect in heterostatic action is expressed, as follows:

$$F' = F + \Sigma \frac{P}{Q_n - P} F^{(n)} \ldots\ldots\ldots\ldots\ldots\ldots (156)$$

Formula (156) may be interpreted as a case of Formula (75). It is of a form similar to Formula (12). In applying Formulas (155) and (156) to the present case of flexure, the index, n, should be replaced by a double index such as m, n. The writer will confine himself to the cases in which $m = n = 1$ gives the smallest critical value, Q, of the astatic load, P. The result is that Formula (156), with the separate terms in Formula (154) substituted for $F^{(n)}$, is a series which begins to converge rapidly immediately after the term corresponding to $m = n = 1$. When the higher terms in the summation in Formula (156) are omitted, that is, when only the first term, which corresponds to $m = n = 1$, is included, the following approximate value is found for the term which must be added to the moment, M_b, in Formula (154) when the end loads, P, are introduced in addition to the already existing surface load, w:

$$M_b'' = \frac{16}{\pi^4} \frac{w\,b^2}{(\alpha^2 + 1)^2} \frac{P}{Q - P} \ldots\ldots\ldots\ldots\ldots\ldots (157)$$

Formula (157) is the same as Formula (30) in Article 10.

The case of the double system of crossing-beams for which formulas were given in Article 11, is analyzed in a similar way. The orthostatic flexure due to the distributed load, w, alone depends on the differential equation:

$$\frac{\partial^4 z}{\partial x^4} + \frac{\partial^4 z}{\partial y^4} = \frac{w}{EI} \ldots\ldots\ldots\ldots\ldots\ldots (158)$$

which can be integrated by the same method as that used in integrating Formula (150), in the case of the rectangular slab. By means of a trigonometric series similar to Formulas (151), (152), and (153), the following value is found for the moment at the center:

$$M = \frac{16\,w\,b^2}{\pi^4} \sum_{1,3\,..}^{m} \sum_{1,3\,..}^{n} \frac{-(-1)^{\frac{m+n}{2}}\,m}{n\,(m^4 + n^4)} = 0.077\,wb^2 \ldots (159)$$

The value, $0.077\ wb^2$ was indicated by Formula (36) in Article 11. The first term in Formula (159) is as follows:

$$\frac{8}{\pi^4}\ wb^2 = 1.07 \times 0.077\ wb^2 \ldots\ldots\ldots\ldots\ldots\ldots (160)$$

It is assumed again that the lowest critical astatic load, Q, corresponds to $m = n = 1$. By substituting Formula (160) for the first term, $F^{(n)}$, in the summation in Formula (156), and omitting the remainder of the terms which, in the case considered, may be assumed to be very small, the following approximate expression is found for the moment at the center in the heterostatic action which is caused by the combined influence of the end loads, P, and the surface load, w:

$$M' = 0.077\ wb^2 \left(1 + 1.07 \frac{P}{Q - P}\right) \ldots\ldots\ldots\ldots (161)$$

Formula (161) is the same as Formula (37) in Article 11.

VI.—Summary.

32.—The investigation deals with structural actions in which the stresses are not proportional to the loads, although the proportional limit of the material has not been exceeded and the deflections remain small. A number of examples of such actions have been mentioned in the Introduction, in Article 1. A simple example is that of the buckling of a slender column under an axial load. A central axial load acting alone on a straight homogeneous column produces "astatic" action; a transverse load acting alone would produce "orthostatic" action; while a combination of the two loads produces a "heterostatic" action. The three terms—orthostatic, astatic, and heterostatic—as applying to structural actions in general, were introduced in Article 1, where orthostatic action is characterized by proportionality of the stresses to the loads; astatic action by the neutral elastic equilibria; and heterostatic action by the combined action of loads which separately would produce orthostatic or astatic action. In the Bibliography (Part VII), a great number of previous investigations of astatic actions are listed.

The present investigation deals with astatic action, and, in particular, with heterostatic action. The study of the latter was made possible by the introduction of a set of "astatic parameters" (Article 15), which were found to have peculiar properties, in particular that of "orthogonality". The general theory is given in Part IV. The use of the astatic parameters leads to simple and general solutions of otherwise complex and difficult problems. The main results of the theory are expressed in Formulas (59), (109), (74), and (97), for the parameters, and Formulas (75), (101), and (102), for the effects in general in heterostatic action.

Although the general theory makes use of the principle of least action, which is expressed in energy equations, certain specific cases of the more simple kind are solved more easily by the direct statical principles which are expressed in differential equations of equilibrium. In Part V, differential equations are set up, which represent cases of straight columns (Formula (138)), curved members (Formula (143)), and slabs (Formulas (147) and

(150)). Special solutions of these equations lead to the specific cases for which formulas are given in Parts II and III.

Part II deals with columns carrying axial and transverse loads at the same time. The solutions depend on the general Formula (12), which is a special case of the Formulas (75), (101), and (102) which are derived in Part IV. The results are found in terms of infinite series, such as Formula (7), but these series appear to be rapidly convergent in the cases investigated, and they can be replaced by approximate formulas with only a few terms in each. Such approximate formulas are given in Table 1, Article 4, and are so simple that they may be used readily by designers of structures. Numerical and graphical results are given in Article 6. In Part III, formulas are given for buckling of slabs and systems of crossing-beams. These cases, also, are special cases to which the results of the general analysis in Part IV may be applied directly.

VII.—Classified Bibliography.

The Bibliography which follows, does not attempt to include references dealing exclusively with straight columns which have constant cross-sections and which carry axial end loads only. This case is treated more or less extensively in almost every textbook on Mechanics of Materials. It is sufficient for the present purpose to point to Euler's analysis of columns, which dates back to 1757, and to the progress which has been marked by the introduction of such formulas as those of Rankine, the parabolic, and the straight-line formulas. A great deal of experimental work has been completed in this field. The deviations from Euler's formula are usually explained by the inevitable presence of small initial eccentricities. In fact, some of the texts* interpret the formulas mentioned as approximate representations of the secant formula which applies to eccentric loading, and which may be used when some definite "equivalent initial eccentricity" has been introduced.

One work on straight columns, of a comparatively recent date, is mentioned in the Bibliography. It is the investigation of columns by Kármán (Section *B*). Kármán's experiments and analysis are noteworthy due to the light which they throw on the influence of stresses above the proportional limit. This matter is important with reference to the shorter columns. Among the other works mentioned in the Bibliography particular attention is called to S. Timoshenko's treatment of a great variety of cases of elastic buckling (Section *A*), and also to the analysis of cylindrical shells by Goupil (Section *G*), in which a special case of the general formula for heterostatic action is derived. It is in the nature of the matter that the following Bibliography cannot lay any claim to completeness.

Section *A*.

This section includes discussions dealing with a variety of cases of buckling or with cases of a general nature.

BRYAN, G. H. On the Stability of Elastic Systems. Cambridge, *Proceedings,* Philosophical Soc. (1888), pp. 199-210.

FOPPL, A. and L. Drang und Zwang, v. 2 (1920), pp. 315-390.

* See, for instance, J. E. Boyd, "Strength of Materials," 1917 Edition.

HENCKY, H. Habilitationsschrift. Darmstadt, 1920.

KARMAN, TH. v. Festigkeitsprobleme im Maschinenbau. Encycl. der Math. Wiss., IV, 27 (1910). See in particular pp. 370-385.

LOVE, A. E. H. Mathematical Theory of Elasticity. Second Edition. Cambridge, 1906. See Index, p. 550, under "Stability".

LORENZ, H. Technische Elastizitätslehre. 1913, pp. 314-365.

OSTENFELD, A. Teknisk Elasticitetslære. Third Edition. Copenhagen, 1916, Chapters V and VII.

RODE. Beitrag zur Theorie der Knickerscheinungen. *Eisenbau* (1916), pp. 121-131, 210-218, 299-305.

——Ueber Schwingungen. *Eisenbau* (1917), pp. 250-258.

SOUTHWELL, R. V. On the General Theory of Elastic Stability. *Philosophical Transactions,* Royal Soc., Lond. (A), v. 213 (1913), pp. 187-244.

TIMOSHENKO, S. Sur la Stabilité des Systèmes Elastiques. *Annales des Ponts et Chaussées* (1913), III, pp. 496-566; IV, pp. 73-132, and V, pp. 372-412.

——Einige Stabilitätsprobleme der Elastizitätstheorie. *Zeitschrift für Math. u. Phys.,* v. 58 (1910), pp. 337-385.

——Also, works in Russian (Yearbooks of the Polytechnical Institutes of Petrograd and Kiev, 1906, etc.).

Section *B*.

This section includes references to column action or buckling in which stresses beyond the proportional limit are considered.

BRIDGEMAN, P. W. The Collapse of Thick Cylinders under High Hydrostatic Pressure. *Physical Review* (1912), pp. 1-24.

KARMAN, TH. von. Untersuchungen über Knickfestigkeit. *Mitteilungen* über Forschungs-*arbeiten auf dem Gebiete des Ingenieurwesens,* v. 81 (1910), 44 pp.

SOUTHWELL, R. V. The Strength of Struts. *Engineering,* v. 94 (1912), pp. 249-250.

Section *C*.

This section includes the more general cases of straight columns, characterized as follows: The axial loads may be transferred at other points than at the ends; or, the cross-section may vary; or, there may be transverse supports at intermediate points; the supports may be rigid or elastic, concentrated, or distributed.

BAIRSTOW, L., and E. W. STEDMAN. Critical Loads for Long Struts of Varying Section. *Engineering,* v. 98 (1914), pp. 403-404.

BLASIUS, H. Träger kleinster Durchbiegung und Stäbe grösster Knickfestigkeit bei gegebenem Materialverbrauch. *Zeitschrift für Math. u. Phys.,* v. 62 (1914), pp. 182-197.

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CASE, J. An Approximate Graphical Treatment of Some Strut Problems. *Engineering,* v. 106 (1918), pp. 699-700.

ELWITZ. Die Knickkraft in Stäben mit sprungweise veränderlichem Trägheitsmoment. *Zentralblatt der Bauverwaltung* (1917), pp. 516-520, 525-528.

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INOKUTY, A. Column of Uniform Strength. *Journal,* Coll. of Eng., Imperial Univ. of Tokio, v. 5, No. 8 (1914), pp. 223-244.

KAYSER, H. Beziehungen zwischen Druckfestigkeit und Biegungsfestigkeit. *Zeitschrift* des Ver, deutscher Ingenieure (1917), pp. 92-97, 124-127.

ONO, A. On the Stability of Long Struts of Variable Section. *Memoirs,* Coll. of Eng., Kyushu Imperial Univ. v. 1, No. 5 (1919), pp. 395-406.

VIANELLO, LUIGI. Graphische Untersuchung der Knickfestigkeit gerader Stäbe. *Zeitschrift* des Ver. deutscher Ingenieure (1898), pp. 1436-1443.

WALLACE, WM. Theory of Taper Columns. *Engineering,* v. 94 (1912), pp. 831-832.

ZIMMERMAN, H. Der gerade Stab mit stetiger, elastischer Stützung und beliebig gerichteten Einzellasten. *Sitzungsberichte* der k. preussischen Akad. der Wissenschaften (1905), pp. 898-912.

——Der gerade Stab auf elastischen Einzelstützen. *Ibid* (1907), pp. 235-244.

——Die Knickfestigkeit des geraden Stabes mit mehreren Feldern. *Ibid* (1909), pp. 180-212.

* An error of computation in Greenhill's paper is corrected by Timoshenko, *Annales des Ponts et Chaussées* (1913), III, p. 514.

Section *D*.

In this section references are given to buckling in which twisting couples are active: Bending of the center line of a shaft into a spiral shape by twisting couples at the ends or by such couples combined with compression; tipping or bending out sidewise of a high narrow beam loaded by forces which are initially in the plane of maximum rigidity (plane of symmetry containing the largest dimension of the cross-section).

GREENHILL, A. G. On the Strength of Shafting when Exposed Both to Torsion and to End Thrust. *Proceedings*, Inst. of Mech. Engineers (1883), pp. 182-209.

PRANDTL, L. Kipp-Erscheinungen (Thesis in Munich). 1899. 75 pp.

Section *E*.

References are made herein to columns or compression members built up as trusses or frames.

AMERICAN SOCIETY OF CIVIL ENGINEERS. Final Report of Special Committee on Steel Columns and Struts. *Transactions*, Am. Soc. C. E., v. LXXXIII (1919-20), pp. 1583-1688.

ELWITZ, E. Berechnung der Knickkraft gegliederter Stäbe. *Zeitschrift*, des Ver. deutscher Ingenieure (1919), pp. 168-170.

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MÜLLER-BRESLAU, H. Ueber exzentrisch gedrückte gegliederte Stäbe. *Sitzungsberichte* der k. preussischen Akademie der Wissenschaften (1910), pp. 166-181.

Section *F*.

Side stiffness of trusses and arches, particularly the side stiffness of bridges of the pony-truss type are referred to in this section.

ENGESSER, FR. Die Zusatzkräfte und Nebenspannungen eiserner Fachwerkbrücken. v. 2 (1893), pp. 105-157.

KEELHOFF. La stabilité des membrures comprimées des ponts métalliques. *Annales des Ponts et Chaussées* (1920), II, pp. 193-231.

MULLER-BRESLAU, H. Graphische Statik, II, 2.

OSTENFELD, A. Teknisk Statik II. Second Edition. Copenhagen, 1913, pp. 483-492.

——Aabne Broers Sidestivhed. *Ingeniören*, Copenhagen (1914), pp. 475 *et seq.*; (1918), pp. 543-549, 577-582.

——Die Seitensteifigkeit offener Brücken, Beton und Eisen. 1916, pp. 117-120, 147-148.

YASSINSKY. Works in Russian, see the previously quoted Memoir by Timoshenko. *Annales des Ponts et Chaussées* (1913).

ZIMMERMAN, H. Die Knickfestigkeit der Druckgurte offener Brücken. *Zeitschrift für Bauwesen* (1910), pp. 329 *et seq.*; also, an enlarged reprint, Berlin (1910), 53 pp.

——Der Stabeck auf elastischen Einzelstützen. *Sitzungsberichte* der k. preussischen Akad. d. Wiss. (1907), pp. 326-338; also, the works of Zimmerman mentioned under Section *C*.

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* The empirical formula proposed by Gérard covers the cases of shorter as well as longer spans and panel lengths. The formula indicated by Lossier gives values which agree rather well with results of Timoshenko's analysis. The latter deals primarily with the cases of long spans. It is maintained by Mathieu that certain tests indicate Gérard's formula as sufficiently safe, and Lossier's formula as in some cases too much on the safe side when the spans are not particularly large.

Section *G*.

This section includes references to circular ring resisting forces in its plane, particularly a uniform outside pressure, and cylinders resisting forces perpendicular to their axes, particularly a uniform outside pressure.

BELSETSKY, STANISLAUS. De la stabilité d'équilibre dans un cas particulier de pièce courbe. *Comptes Rendus*, v. 156 (1913), pp. 1056-1058.

BRESSE, J. A. CH. Cours de mécanique appliquée. Paris, 1859, pp. 326 *et seq.*

BRYAN, G. H. Applications of the Energy Test to the Collapse of a Long Thin Pipe under External Pressure. Cambridge, *Proceedings*, Philosophical Soc. (1888), pp. 287-292.

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——**and M. L. CARR.** Resistance of Tubes to Collapse. *Ibid, Bulletin 5* (1906).

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——The Resistance of Tubes to Collapse. British Assoc. for the Advancement of Science (1913), pp. 213-224; gives a bibliography on collapse of cylinders, with notes.

ENGER, M. L., and F. B. SEELY. Vents on Steel Pipe Lines. *Engineering Record*, v. 69 (1914), pp. 594-596.

FORCHHEIMER, PH. Zur Festigkeit weiter Rohre. *Zeitschrift* d. Oesterr. Ing. u. Arch. Vereines (1904), pp. 133-137, 149-152.

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HURLBRINK, E. Festigkeitsberechnung von röhrenartigen Körpern. *Schiffbau*, v. 9 (1908), pp. 517-523, 557-563, 599-607, 637-643.

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MISES, R. von. Der kritische Aussendruck zylindrischer Rohre.* *Zeitschrift* des Ver. deutscher Ingenieure (1914), pp. 750-755.

SLOCUM, S. E. The Collapse of Tubes under External Pressure. *Engineering*, v. 87 (1909), pp. 35-37.

SOUTHWELL, R. V. On the Collapse of Tubes by External Pressure. *Philosophical Magazine* (6) (1913), v. 25, pp. 687-698; v. 26, pp. 502-511.

USINGER, P. Knickerscheinungen bei Zylindern und Ringen. *Dingler's Journal* (1917), pp. 85-89.

Section *H*.

Buckling of a cylindrical shell due to loads in the direction of the axis of the cylinder (tubular struts), are referred to in this section.

LORENZ, R. Die nicht achsensymmetrische Knickung dünnwandiger Hohlzylinder. *Physikalische Zeitschrift* (1911), pp. 241-260.

Section *J*.

This section includes references to slabs and plates, including the case of the buckling of the web of a girder due to shearing forces.

BRYAN, G. H. On the Stability of a Plane Plate under Thursts in Its Own Plane. *Proceedings*, London Math. Soc., v. 22 (1890), pp. 54-67.

MOORE, H. F., and W. M. WILSON. Strength of Webs of I-beams and Girders. Univ. of Illinois, Engineering Exp. Station, *Bulletin 86* (1916).

NADAI, A. Ueber das Ausbeulen von kreisförmigen Platten. *Zeitschrift* des Ver. deutscher Ingenieure (1915), pp. 169-174, 221-224.

REISSNER, H. Ueber die Knicksicherheit ebener Bleche. *Zentralblatt der Bauverwaltung*, v. 29 (1909), pp. 93-96.

SOMMERFELD, A. Ueber die Knicksicherheit der Stege von Walzwerkprofilen. *Zeitschrift für Math. u. Physik.*, v. 54 (1906-07), pp. 113-153, 318-324.

TIMOSHENKO, S. On the Stability of Plates in Compression. Polytechnical Inst., Kiev, *Annals* (1907).

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*Gives formula applying to pipes with stiffening rings.

Section *K*.

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CHREE, C. The Whirling and Transverse Vibrations of Rotating Shafts. *Philosophical Magazine* (6), v. 7 (1904), pp. 504-542.

DUNKERLEY, S. On the Whirling and Vibration of Shafts. *Philosophical Transactions,* Royal Soc. (London), A, v. 185, I (1894), pp. 279-360.

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GREENHILL, A. G. See paper quoted under Section *D*.

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WEBB, H. A. The Whirling of Shafts. *Engineering,* v. 104 (1917), pp. 455-456, 483-485, 513-514.

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PAPERS AND DISCUSSIONS

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THE FLOOD OF JUNE, 1921, IN THE ARKANSAS RIVER, AT PUEBLO, COLORADO

Discussion*

By Messrs. Arthur O. Ridgway, R. G. Hosea, and George G. Anderson.

Arthur O. Ridgway,† M. Am. Soc. C. E. (by letter).‡—For the sake of clarity in discussing this admirable paper, any comment should conform in sequence to the different sections outlined therein.

History of Former Floods.—A thorough study of all available sources of information indicates that no flood of the magnitude of the 1921 deluge had occurred at the site of Pueblo for at least 100 years previously. Indians and early settlers left stories of a great flood in 1844, but, on investigation, this flood was found to be undoubtedly the result of a tremendous snow which fell in the valley in the early spring and did not entirely disappear for several weeks afterward. The evidence collected shows that although probably of greater volume, because of vastly longer duration, it could not have exceeded in height the flood of 1864. Evidence is also at hand to the effect that, for many years prior to 1844, no such flood as that of June, 1921, could have occurred. That the flood of 1894 substantially exceeded the one which occurred in 1864 has been definitely determined, and that the recent flood of June, 1921, was far greater than the 1894 deluge is not open to question.

In connection with these several floods, the evolution of the channel of the Arkansas River through the site of Pueblo is most interesting and effects a valuable contribution to the information necessary in planning preventative or protective measures. The authors state that at the time of the flood of 1894, "Pueblo had little or no river protection, and the Arkansas meandered through the city, cutting its banks and changing its course." This is not strictly correct.

Definite data are at hand, which show the various channels of the Arkansas from 1872 to the present time. It is more than probable that the position

* This discussion (of the paper by James Munn and J. L. Savage, Members, Am. Soc. C. E., published in September, 1921, *Proceedings*, and presented at the meeting of October 5th, 1921), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Denver, Colo.

‡ Received by the Secretary, September 23d, 1921.

of the channel in 1872 reflects conditions extant during the flood of 1864. This channel had a greater width than any subsequent channels through the main part of the town, and, notwithstanding the fact that the course was very sinuous, its carrying capacity was no doubt quite comparable with that of later channels.

Some time between 1872 and 1881, whether by encroachment of the growing city on the older channel, or by actual channel improvement work, as to which need not now be determined, the channel through the town underwent a decided straightening and narrowing. Its position in the lower part of the city was entirely changed, eliminating a serious detour which existed in the older channel. Between 1881 and 1889, the straightening continued to some extent. The floods of 1889 gave an impetus to channel improvement work, and, by 1894, the channel was practically confined through the built-up part of the city. At the time of the flood of 1894, additional levee work was in progress, especially in the vicinity of the West Fourth Street Viaduct. Subsequently, the levees were completed through the city, the bridges were raised, and the channel otherwise was improved to carry a flood equal in magnitude to that which occurred in that year, the volume of which was determined at the time to be about 40 000 sec-ft. The programme of these improvements was not entirely completed until about 1898, since which time little or no additional protective work has been effected. Since the completion of the channel improvements, floods of considerable magnitude have occurred at times, but on no occasion has there been any serious damage and apparently no serious menace. The flood of June, 1921, so far exceeding any of which there is a recorded history, was totally unexpected and no doubt will be included in the list of great American disasters.

Description of June, 1921, Flood.—The Arkansas River at Pueblo quickly receded from flood crest on the night of June 2d, 1921, to nearly normal stage by 8.00 A. M., June 3d, without leaving any ill effects on the channel, and remained at that stage until 5.00 P. M., of June 3d; therefore, only the events which transpired subsequent to that time are involved.

The Deputy State Engineer has fixed the time of overflowing of the channel at the Main Street Bridge, where the river gauge is located, at 8.45 P. M., June 3d, and has computed the discharge of the stream at that time to have been 45 000 sec-ft., with a gauge height of 18.14 ft. The floor of the Union Depot has an elevation equivalent to a gauge reading of 17.61 ft., or 0.53 ft. lower than the coping of the channel walls at the Main Street Bridge. Throughout the night of June 3d, a log was kept in the Union Depot of the various stages of the water, both during its rise and recession. This log, perhaps the only one in existence compiled from actual measurements co-ordinated with accurate and frequent time intervals, shows the stages and duration of the overflow peak, as given in Table 15.

The figures in Table 15 clearly indicate the remarkably short duration of the overflow and the very rapid rise of the river, especially between the hours of 11.31 and 11.46 P. M., when the rate was 1 ft. for each 5-min. period. The subsidence, although phenomenally rapid, was slightly slower. The peak occurred at 11.55 P. M., and not at 1.00 or 1.30 A. M., as heretofore assumed.

At no time during the period of either rise or fall does the stage appear to have remained stationary; even the highest stage was only momentary. In view of the position of the Union Depot with respect to the river channel, and considering all the other relevant circumstances, there seems to be no sound reason for not considering the rise as actually representing conditions on the Main Street gauge. The subsidence at the Union Depot, although in all probability lagging slightly, ought to be regarded as exactly parallel with that of the channel. The crest at the Union Depot reached a gauge of 27.36 ft., as compared with the reported peak gauge of 24.66 ft., taken from a high-water mark near the Main Street Bridge. Reference of the Union Depot readings, observed in comparatively quiescent water during the progress of the overflow, to the gauge datum has been confirmed by spirit leveling, and there seems to be no question as to their accuracy.

TABLE 15.—FLOOD STAGES AT UNION DEPOT, PUEBLO, COLO., JUNE 3D AND 4TH, 1921.

Time of rise (read up), June 3d.	Gauge height extended, in feet.*	Time of fall (read down), June 4th.
11.55 P. M.	27.36	12.01 A. M.
11.54 P. M.	27.00	12.08 A. M.
11.46 P. M.	26.00	1.03 A. M.
11.41 P. M.	25.00	1.22 A. M.
11.36 P. M.	24.00	1.37 A. M.
11.31 P. M.	23.00	2.02 A. M.
11.10 P. M.	22.00	2.28 A. M.
10.43 P. M.	21.00	3.01 A. M.
10.12 P. M.	20.00	3.47 A. M.
9.43 P. M.	19.00	5.02 A. M.
9.05 P. M.	18.00	7.15 A. M.

* Floor of Union Depot = gauge height of 17.61 ft.

There is little doubt but that the river had fallen to the channel overflow point by about daylight. This is confirmed in a general way by the log at the Union Depot, for with the more than 1 ft. of mud deposited on the depot floor, the observer could scarcely determine just when the water left the structure. In determining the volume of the flood, the important fact for consideration is that notwithstanding the river had fallen to the overflow point at the Main Street Bridge, substantial volumes were flowing, with no insignificant velocities, through other parts of the city from the broken levees near the West Fourth Street Viaduct.

The flood in the Fountain River can now be said with a considerable degree of certainty to have joined the Arkansas River on the eastern outskirts of Pueblo at about 3.00 A. M., on June 4th. This was 3 hours after the peak was reached by the Arkansas, or when that stream had fallen to a stage within 3 ft. of the overflow height. The discharge of the main stream, therefore, could not have been more than 60% or 70% of the peak flow, whatever that was, when it was joined by the Fountain River which from an accurate flood section is now estimated to have carried a maximum of about 40 000 or 45 000 sec-ft.

In a supplemental report, R. G. Hosea, Deputy State Engineer, gives some results of his field investigation of the tributaries of the Arkansas River east of Pueblo, together with a computed rate of discharge of the main stream near La Junta, Colo., 65 miles east of Pueblo. These results are as follows:

St. Charles	50 000 sec-ft.
Chico	20 000 " "
Huerfano	5 000 " "
Arkansas at La Junta	180 000 to 200 000 " "

As flood crests in the tributaries were not synchronous with the main stream, nor with each other, it is more than probable that the maximum discharge of the river at La Junta did not exceed 200 000 sec-ft.

At 9.30 A. M., on Sunday, June 5th, the Schaeffer Dam, an earthen structure, 90 ft. high, on Beaver Creek and about 11 miles from its junction with the Arkansas, failed with a crash. The impounding reservoir, estimated to contain about 4 000 acre-ft. at the time of failure, was emptied in 30 min. The attendant flood reached Swallows, a point on the river 8 miles below the mouth of Beaver Creek, at 11.30 A. M.; Goodnight, 10 miles farther down, at 1.30 P. M.; and Pueblo, an additional 5 miles, at 2.15 P. M. The itinerary of this wave may be scheduled as shown in Table 16.

TABLE 16.

Place.	Flood arrived.	Lapsed time.	Distance traveled.
Reservoir site	9.30 A. M.		
		2 hours	19 miles
Swallows	11.30 A. M.		
		2 hours	10 miles
Goodnight	1.30 P. M.		
		0 hours 45 min.	4 miles
Pueblo	2.15 P. M.		
Total	....	4 hours 45 min.	33 miles

It is important to note that at its crest the flood, as recorded at the Main Street gauge, did not exceed 28 000 sec-ft. at a gauge height of 13 ft., or more than 5 ft. below the 18.14-ft. gauge of overflow point. In all probability if there had been no breaks in the levees from the flood of June 3d, the channel would have carried the flow from the Schaeffer Reservoir. The most important fact, however, is that notwithstanding the reading on the Main Street gauge was 5 ft. below the overflow point, the water was several inches above the floor of the Union Depot, and probably reached a gauge height there of about 18.00 ft., the overflow point at Main Street. This is direct confirmation of a previously mentioned observation to the effect that when the river had fallen to its channel overflow point at Main Street, in the early morning of June 4th, with a rated discharge flow of 45 000 sec-ft., large volumes of water were flowing through other parts of the city from breaks in the levees near the West Fourth Street Viaduct. Although the total volume from the Schaeffer Reservoir was relatively small and effected no serious

damage at Pueblo and points farther east, yet, due to the release in the body, the damage inflicted in the valley west of Pueblo was far greater than in the previous flood, and left a real "valley desolate".

Flood Loss.—The authors are to be congratulated in assembling even preliminary figures on this most illusive factor of property loss, and there seems to be no warrant for suggesting change in the amount submitted. The figures are of inestimable value in planning protective or preventative measures and are an index of the justifiable extent to which such measures can be recommended. Doubtless these amounts, together with the possibility of a future occurrence of a flood of equal or greater magnitude, will be weighed against the cost of assured protection. It is too bad, though, that mention was not made of the most vital factor in the whole matter. More than 150 lives were lost—the exact number can never be known. Thus far, 78 bodies have been recovered. Men might ponder at great length as to the justifiable investment to guard against property loss, but if the people in this part of the Arkansas Valley must return to their industrial pursuits, and it is assumed that they must, there is no warrant for hesitancy. The futility of placing monetary value on a human life should not restrain the engineer as a true conserver from placing personal safety in industry far above all things else.

Estimated Peak Flow and Volume.—The peak flood of the Arkansas River through the City of Pueblo was in all probability about 100 000 sec-ft., as first estimated by Mr. Hosea. This figure will be confirmed or corrected when the investigations now in progress are completed. The channel had a flood capacity of about 40 000 sec-ft., without any margin of safety. Improvements necessary to insure such capacity with a reasonable degree of safety could be effected at nominal expense. It would seem, therefore, that in considering protective measures involving substantial investment, that provision is necessary only for those floods of a greater discharge rate than 40 000 sec-ft. The volume in the Arkansas, passing through the city in excess of a discharge rate of 40 000 sec-ft. could not have been more than 30 000 acre-ft. The Fountain River, flooding a safe channel with a capacity of 25 000 sec-ft., for 12 hours, with a peak discharge of 45 000 sec-ft., would have a volume of about 10 000 acre-ft. without channel provision. In other words, if 30 000 acre-ft. had been held back on the Arkansas for a few hours and 10 000 acre-ft. on the Fountain, no disaster would have occurred at Pueblo on the night of June 3d.

It does not appear that the combined maximum flow of the Arkansas and Fountain of 65 000 sec-ft., together with the accretions of the tributaries east of Pueblo, would work any hardship on the lower valley. Prior to the June flood, a peak flow greater than 40 000 sec-ft. had not passed Pueblo in the Arkansas for a period of 100 years or more, if historical deductions are correct. The average daily discharge for a period of 11 years—1910-20—was 770 sec-ft.

Drainage Area and Run-Off Data.—It is true that the part of the Arkansas Basin between Canon City and Pueblo supplied nearly all the run-off in the flood of June 3d, yet by no means can the drainage basin of 3 047 sq. miles above Canon City be ignored in considering protective measures. Notwithstanding the fact that the greatest discharge at the Hanging Bridge in the

Royal Gorge since the construction of the railroad in 1880 has probably not exceeded 9 000 sec-ft., Grape Creek, which flows into the Arkansas just below the Grand Canyon, has a formidable drainage area of 483 sq. miles and is subject to excessive floods. The railroad branch in the canyon occupied by this stream extending from Canon City to Silver Cliff was several times seriously damaged by floods, and, in 1889, after only nine years of operation, was so nearly destroyed that it was permanently abandoned.

Table 17 shows the daily and total precipitation measured by various observers at points in the Fountain and Arkansas Basins during the storms of June 2d-6th, 1921. Attention is directed to a previous error in the Colorado Springs precipitation for June 3d which is corrected herein.

TABLE 17.—INCHES OF RAINFALL, JUNE 2D-6TH, 1921.
(All gauges read at 6 P. M., except as noted.)

Station.	Drainage basin.	DAY.					Total. 2d-6th.
		2d.	3d.	4th.	5th.	6th.	
Monument	Fountain	....	......	2.90	0.82	0.05	3.77
Colorado Springs	"	....	0.50	4.40	1.26	0.42	6.58
Fremont	"	....	2.53	2.61	1.43	0.48	7.05
Lake Moraine	"	....	0.65	3.68	1.40	0.18	5.91
Victor*	Oil Creek	....	0.03	2.08	1.55	0.37	4.03
Canon City†	Arkansas	....	0.30	2.35	0.75	0.40	3.80
Florence†	"	....	0.99	3.31	2.47	0.13	6.90
Penrose†	Beaver	....	......	7.00	2.00	1.50	10.50
Pueblo‡	Arkansas	1.94	1.64	1.45	1.12	0.09	6.15
Buena Vista	"	....	......	0.90	0.42	....	1.32
Leadville	"	....	......	0.16	0.49	0.71	1.36
St. Elmo	Chalk Creek	....	......	0.63	0.55	0.38	1.56
Hayes Ranch	Osteen Creek	....	10.00	Friday evening and night.			10.00
Hobsin Ranch	Arkansas	....	6.50	" " " "			6.50
Teller	Turkey Creek	....	7.50	" " " "			7.50
Boggs Flat	Boggs Creek	....	14.00	" " " "			14.00
Higgins	Eight Mile	....	12.00	" " " "			12.00

* Gauge read at 4.00 P. M.
† " " " 8.00 A. M.
‡ " " " midnight.

It is inconceivable that the heavy precipitation of Friday evening and night could extend over any considerable area, and equally irrational to apply the averages deduced therefrom over the entire drainage basin of either the Fountain or Lower Arkansas Rivers. An incontrovertible fact of long standing is that by far the greater volume of precipitation in the Colorado Arkansas is the result of violent rain storms. There should be no hesitancy in saying that the basin, especially the lower part, is infested with cloudbursts, for no other nomenclature better fits the observed phenomena.

Grouped according to elevation, these stations furnish the results shown in Table 18.

It will be observed from Table 18 that, for this particular series of storms, the precipitation varies inversely with the altitude, a fact contrary to the commonly accepted law which doubtless will apply only to precipitation from general storms. Even in the mean annual precipitation, there is a slight vagary in this law in the Upper Arkansas Basin. No reason is advanced as

to why the probable maximum precipitation in 72 hours should be exactly 140%, rather than any other multiple of the greatest precipitation of record; neither is it clear that the figures for percentage of run-off will apply to this particular region. In the 11 years, 1910-20, the ratio of mean annual run-off to mean annual precipitation for the basin above Canon City was 23%, and for the entire basin above Pueblo 19 per cent. On the other hand, it is easy to conceive that the ratio from cloudbursts, in some of the tributaries with excessive gradients, might closely approach 100 per cent. Apparently, the law of averages will not apply in this case, except in a very limited manner; rather should the analysis consist in a study of each individual tributary. For example, it is known that Oil Creek, with a drainage area of 416 sq. miles, never floods frequently or excessively, while, in contrast, Rock and Peck Creeks are subject to frequent floods. Hardscrabble has idiosyncrasies well worthy of analysis. Other tributaries have similarly developed certain characteristics, the permanency of which has not yet been assailed. Doubtless, the position of the confining mountain ranges, their altitude, and their relation to each other, as well as to the Great Plains, have a great deal to do with the case.

TABLE 18.—Average Inches of Rainfall, Stations Above Pueblo, Colo., in 72 Hours, June 2d-6th, 1921.

Elevation, in feet.	Number of stations.	Average inches of rainfall, 72 hours.
4 600 to 5 000	6	9.17
5 000 to 6 000	4	6.94
6 000 to 7 000	1	3.77
Above 7 000	6	3.49
Total..................	17	6.41

It may be possible to construct from past records, combined with personal experiences and accumulated data, a synthetic programme of a probable maximum flood at Pueblo. The apparently insurmountable obstacle in this programme will be the occurrence and intensity of cloudbursts, the forecasting of which has thus far baffled all attempts at solution.

Flood Control.—The answer to the problem seems to lie in some scheme of flood-regulating reservoirs. This is by far the safest plan, for, regardless of the magnitude of the flood peak, discharge through the channel will be limited to its capacity. The only further concern will be in ascertaining the volume of maximum flood which it will be necessary to hold back, and this factor is more susceptible of definite determination than the peak flow of any imaginary flood.

Obviously, the channel through the city should be as large as ultimate economy will permit. The larger the city channel, the smaller will be the required detention. This is a distinct advantage in that it will reduce the cost of maintaining reservoir capacity against the deposit of enormous quantities of débris brought down by the torrential flood velocities of the river and

its tributaries. On the other hand, the city channel ought to be as small as possible, for, otherwise, property suited for intensified use would be devoted to only occasional service during extraordinary floods. An infrequently used flood channel, many times larger than normal flow requirements, would constitute waste land in the heart of the city and impose extra burdens on the daily conduct of city affairs. Again, a city channel of maximum flood capacity would be no safeguard to life and property in the valley east of Pueblo, whereas confinement of the river within minimum and permanent limits would permit the cultivation of land otherwise unproductive.

In the interest of conservation, it would seem, therefore, to be necessary first, to determine the size of the channel which the value of property both urban and rural can consistently bear, and weigh this cost with that of reservoirs to supplement the channel in passing the floods. The only unknown quantity in the problem is the volume and peak flow of the possible maximum flood. The suggestion of a rainfall 40% in excess of the two greatest storms on record, distributed over the water-shed according to altitude and running off at assumed rates, is not entirely satisfactory. There is no certainty that the records of the 1894 flood tell the whole story, and it is quite certain that the records used for the 1921 flood do not. Moreover, the altitude distribution over the whole water-shed of the few available averages and the assumed percentages of run-off do not seem wholly applicable to the Arkansas Basin. Here, the precipitation is spotted, storms are greatly concentrated in area, flood rainfall is extreme in intensity, and run-off is widely variant.

As the concern in the maximum peak discharge is only for the purpose of providing requisite channel capacity without control reservoirs, it may be that the suggestion of 168 000 sec-ft., as crest discharge in the Arkansas, will answer the purpose. However, it is entirely possible, with the same intensity of storms and slightly different distribution over the tributary basins, to have greater peak flows with smaller volumes than even that which occurred on June 3d. In this phase of the problem, the vital factor is the torrential rains in certain tributaries which might make their peak discharges coincident at the critical point in the river. The difficulty lies in the local concentration of storms in places where no long-time records have been kept. Similarly, it may be that the figures of 322 000 acre-ft. in the Arkansas River represent the greatest possible volume of flood of a few days' duration, but as the method by which they were deduced is open to question, their reliability is not assured.

As previously mentioned, the present channel of the Arkansas River can and did carry 40 000 sec-ft., and can be made to do so with an adequate margin of safety at nominal expense. Apparently, therefore, the detention reservoir capacity should be computed on this basis. The development of the Rock Canyon site to requisite capacity, even with this promise, may not be sound economy, but it does not yet appear to be impracticable to obtain a part of the detention capacity on some of the tributaries. For instance, Dry, Boggs, and Rock Creeks were the worst offenders in the recent flood. Peck Creek is often and Hardscrabble occasionally the culprit, but this time neither was very unruly. The flood duration on all these streams is very brief. From data

collected in the field, the maximum rate of discharge and total volume of flood for the three first-named streams may be approximated as follows:

	Second-feet.	Acre-feet.
Dry Creek	17 000	7 000
Boggs Creek	33 000	13 500
Rock Creek	36 500	15 000

The channels of these streams and the river below their conjunction therewith will carry substantial flows without damage, and apparently reservoirs of adequate capacity for their control would be quite practicable. On investigation other tributaries will doubtless yield even greater promise of advantage.

It is entirely practicable, although somewhat expensive, to remove the railroad tracks and the Bessemer Canal from any reservoir basin at the Rock Creek site. The suggestion of leaving these works within the basin to be subject to occasional submergence as an economic measure is not at all clear. Aside from that incurred by the failure of the Schaeffer Dam, most of the railroad damage west of Pueblo accrued from floods in the tributaries. Any advantage gained by checking main stream velocities within the reservoir limits would doubtless be trifling in contrast to the enormous loss from inundation and subsequent excessive cost of clearing the submerged works from a heavy deposit of detritus.

The topography of the Fountain River is not as favorable as that of the Arkansas for the construction of retarding reservoirs, but the channel through the town is more susceptible of improvement and enlargement. Any reservoir site on the Lower Fountain River, and there seems to be none of any great promise, will be expensive for the installation of control works, and, perhaps, prohibitive in cost per acre-foot capacity. It is also quite likely that the difficulty of securing satisfactory foundations will militate against the adoption of a detention plan.

A consideration of all the phases of the situation indicates preference for the improvement and enlargement of the channel through the city as the best plan. This view is influenced by the probability that, after a thorough investigation, the peak flow of any future maximum flood will be taken at substantially less than the suggested 110 000 sec-ft. Present information indicates that the probable maximum crest will exceed the 45 000 sec-ft. of June 4th by only a small proportion. However, any plan for flood control on the Fountain River must be co-ordinated with plans for controlling the Arkansas River, in order to obtain the desired results in the valley east of Pueblo.

The authors are to be commended most highly for including even a roughly approximate statement of cost for the various plans suggested. This information will doubtless serve the desired purpose of emphasizing the gravity of the entire situation. With still only very meager data at hand, it is impossible to confirm or revise the estimates. Attention is called to a possibility of reducing the amounts very materially after a more mature investigation as to whether the Fountain peak of 45 000 sec-ft. of June 3d and 4th, was not a probable maximum, or nearly so, and also by basing the Arkansas computations on a channel capacity of 40 000 sec-ft.

The utilization of water accumulated by detention reservoirs for either irrigation or power purposes does not now appear to be entirely practicable. Detention reservoirs must be emptied as speedily as the channel capacity will permit, for in some seasons torrential rains in the water-shed have a pernicious habit of rapid repetition. The storage of large quantities of water to be used slowly could only be accomplished by quickly drawing off the detained volumes into other basins through canals of large dimensions. Still, this feature of the case is a move toward the utmost utilization of natural resources, and it is worthy of some consideration and investigation.

R. C. HOSEA,* ESQ. (by letter).†—Since the Arkansas River flood of June 3d, 1921, two questions frequently asked of the State Engineer's Office, are:

1. What was the peak flow of the flood at Pueblo?

2. What quantity could have been safely carried through Pueblo, in the existing river channel?

The first question is extremely difficult to answer and probably may be estimated more intelligently after a consideration of the second.

Through the main part of Pueblo, the channel of the Arkansas River is approximately 150 ft. wide, between nearly vertical masonry walls 18 ft. high. The State gauging station was located at the Main Street Bridge, where frequent measurements were made, at low-water stages by wading and from the bridge during high water. On June 2d, at 10.00 A. M., the following measurement was made by Messrs. H. D. Amsley and G. C. Price, State Hydrographers: Gauge height, 4.69 ft.; area, 444 sq. ft.; discharge, 2 417 sec-ft.; and mean velocity, 5.44 ft. per sec.

From 6.00 to 6.20 P. M., on June 3d, these men also made a float measurement by timing with stop-watches logs and drift over a 100-ft. course. This measurement is only a rough approximation, since the river rose 1 ft. (from Gauge 10.6 to Gauge 11.6) during this time, and it was impossible to obtain accurate soundings. The depths, therefore, were computed, using 11.2 ft. as the mean gauge height for the period. The velocities thus obtained were 10 ft. per sec. near the side-walls and 15.2 ft. per sec. in the central part of the channel.

The area computed, using 11.2 ft. as the gauge height (and making an assumption as to the amount of scouring out of the channel, as explained later), was 1 505 sq. ft. The mean velocity was 11.8 ft. per sec., based on the following assumptions: Mean velocity = 0.90 of surface velocity; surface velocity for 30 ft. from each wall = 10 ft. per sec.; and surface velocity for 90 ft., central part of channel = 15.2 ft. per sec. This gives a discharge of 17 905 sec-ft.

The channel at the Pueblo Station is subject to scouring during high water and filling again under normal conditions. This condition was particularly apparent before and after the flood of June 3d. A measurement made on June 7th showed: Gauge height, 5.60 ft.; area, 788 sq. ft.; discharge, 6 274 sec-ft.; and mean velocity, 8.0 ft. per sec.

Comparing this measurement with that of June 2d, it is seen that the gauge height was 0.91 ft. higher, corresponding to an increase in area of

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† Received by the Secretary, September 26th, 1921.

0.91 $\times$ 150, or 136.5 sq. ft., while the measured area showed 344 sq. ft. more, leaving an excess difference of 208 sq. ft. caused by scouring. This amounts to an average increase in depth of 1.4 ft. in a channel 150 ft. wide (in one place, the actual increase was slightly more than 3 ft.). In computing the area for the float measurement of June 3d, it was assumed that one-third of this scouring had taken place. A measurement on June 29th, showed: Gauge height, 4.77 ft.; area, 542 sq. ft.; discharge, 2 865 sec-ft.; and mean velocity, 5.38 ft. per sec. This area when compared with that of June 7th, shows that during this time the channel filled an average of 0.8 ft. over the 150-ft. section, and is evidently approaching its condition prior to the flood.

The Main Street Bridge at Pueblo is an old-fashioned Bollman truss, with floor-beams spaced about 10 ft. apart, extending below the lower chord. Levels taken a few days after the flood showed the following gauge heights: Bench-mark on stone coping (top of channel side-wall), Elevation 4875.19, equals 18.18 ft.; sidewalk of bridge, 18.66 ft.; top of floor-beams, 17.68 ft.; bottom of floor-beams, 14.68 ft.; and bottom of stringers, 16.85 ft. The high-water mark on the City Auditorium, adjacent to the bridge, is 4 681.71, or 6.5 ft. above the top of the channel coping.

It is evident that the channel could not safely carry water over a gauge height of 14.68 ft. and at this elevation it would reach the bottom of the floor-beams. However, as an estimate of what might have been carried in the channel, a gauge height of 16.85 ft. has been used as the clearance line of the bottom of the lower stringers of the bridge. Probably the most accurate determination of the flow at this gauge height may be made from a logarithmic discharge curve, based on previous measurements and on the measurements already referred to, since this will be a straight line and can be accurately extended.

At a gauge height of 16.85 ft., such a curve gives a discharge of 45 000 sec-ft. From this, and the computed area (approximately 2 300 sq. ft.) a velocity of 19.5 ft. per sec. is obtained. Extending a velocity curve is not satisfactory, because of discordant values due to shifts. Such an extension, however, indicates a velocity of about 18 ft. per sec. at a gauge height of 16.85 ft. and a corresponding discharge of 41 400 sec-ft.

As a rough check on these values, on June 29th, 1921, a determination of Kutter's n was made for the river channel from the Victoria Avenue Bridge to the Main Street Bridge, a distance of 768.5 ft. At the Victoria Avenue, Union Avenue, and Main Street Bridges, careful levels were taken to the water surface at nine different points on each section. Using the mean of these nine values as the surface elevation at each section, the following results were obtained:

Victoria Avenue, mean water surface	4 662.61
Union Avenue, " " "	4 662.25
Main Street, " " "	4 661.82
Difference in elevation,	
Victoria Avenue to Main Street	0.79 ft.
Distance	768.5
Slope	$\frac{0.79}{768.5} = 0.001 +$

Careful current-meter measurements were made at Main Street and at Union Avenue, and a careful cross-section was taken at Victoria Avenue. The two meter measurements showed a difference of 3.4% (to be expected in cable measurements), and the mean discharge of 2 865 sec-ft. was used. The height on the Main Street gauge was 4.77 ft.

The "hydraulic elements" of the three sections were as given in Table 19.

TABLE 19.

	Victoria Avenue.	Union Avenue.	Main Street.
Area, in square feet	485.7	488.7	512.1
Perimeter, in feet	155.0	155.0	155.0
Discharge, in second-feet	2 865.0	2 865.0	2 865.0
Mean velocity, in feet per second	5.9	5.75	5.38
Hydraulic radius	3.2	3.2	3.5

The mean section computed from the values given in Table 19, has the following elements: Area, 505.5 sq. ft.; discharge, 2 865 sec-ft.; mean velocity, 5.67 ft. per sec.; wet perimeter, 155; $r = 3.26$; and $s = 0.001$.

Solving for Kutter's C and n, these elements give $C = 99.3$, and $n = 0.0182$. Merriman's "Hydraulics", 1906 edition, p. 282, gives for Kutter's n:

$n = 0.20$ for canals in very firm gravel.
$n = 0.017$ " rubble masonry.

This instance is a canal section with a firm gravel bottom and rubble masonry sides, and, as might be expected, n is intermediate between 0.017 and 0.020.

It is found that, for the float measurement of June 3d, 1921, $C = 126$ and $n = 0.016$, using the following elements: Area, 1 505 sq. ft.; mean velocity, 11.8 ft. per sec.; perimeter, 172.0 ft.; $r = 8.75$; and $s = 0.001$.

This is based on the assumption that the slope remains the same for a much higher gauge height and a much greater discharge, which, in turn, assumes a free outflow below. In view of these assumptions and the uncertainty in the measured velocity, it is probable that the value of $n = 0.016$ should only be taken as an indication that n decreases as the flow increases.

If n does decrease at the rate indicated, for a gauge height of 16.85 ft., it should be about 0.014, in which case: Area, 2 300 sq. ft.; discharge, 37 950 sec-ft.; perimeter, 182.5 ft. (approx.).; $r = 12.5$; $s = 0.001$; $n = 0.014$; $C = 147$; and $v = 16.5$ ft. per sec.

In applying three different methods, the three values obtained:

(*a*) 45 000 sec-ft., logarithmic discharge curve;
(*b*) 41 400 " " extended velocity curve;
(*c*) 37 950 " " Kutter's formula;

give a mean discharge of 41 450 sec-ft., or, roughly, 40 000 sec-ft., as the maximum which could pass under the Main Street Bridge.

Peak Flow.—The actual peak flow of the flood of June 3d, 1921, is not known, but an estimate may be made in an intelligent way and thus, perhaps, a figure established within certain reasonable limits. It is not known how much of the excess height (above the top of the river channel) was caused by obstructions at bridges and in the area adjacent to the mouth of the Fountain River. High-water marks show that the river rose about 7 ft. above the top of the stone coping which tops the side-walls of the river channel. If the maximum possible carrying capacity of the channel to the top of the coping is taken as 45 000 sec-ft. and to that is added the quantity which could be carried by an additional 7 ft. in depth (which would be, say, 150 ft. wide by 7 ft. deep by 20 ft. per sec. velocity), thus making a total of 45 000 + 21 000, or 66 000 sec-ft. This assumes, of course, an unobstructed flow in a channel 7 ft. deeper than the present one, but takes no account of the water which flowed outside the present channel, through the town.

Since, of course, this condition of unobstructed flow did not and could not exist at this gauge height, it may be contended that this velocity of 20 ft. per sec. did not exist and that, as the water backed up, its velocity was decreased. No account however has been taken of the quantity of water passing through the town, and it is probable that the above effect must have been more than counterbalanced and that the figure of 66 000 sec-ft. is a minimum estimate of the peak.

On June 19th, 1921, a cross-section of the Arkansas River at a point above the mouth of Dry Creek at the Denver and Rio Grande Mile Post 120 + 4 082 was surveyed by W. A. Balcom, Division Engineer, of the Denver and Rio Grande Railroad. From high-water marks, the area of this section was found to be 8 610 sq. ft., wetted perimeter, 751.6 ft., and the slope of the river was 0.0028 for 1 200 ft. above the section and 0.0022 for 877 ft. below it. Using a mean slope of 0.0025 with these figures and a value of $n = 0.035$, Kutter's formula gives a discharge for this section of 99 876 sec-ft. Using $n = 0.04$ (which seems to be a more reasonable value), the discharge becomes 86 100 sec-ft., about 14% less.

To this must be added the flow of Dry Creek. Late Thursday night, June 2d, 1921, a local cloudburst flooded Dry Creek, and judging from reports, it carried more water then than at the time of the "big flood". Therefore, estimates made from water-marks may give figures for this first flood. The first flood, a large part of which came from the cloudburst on Dry Creek, however, was recorded on the automatic gauge at Pueblo, and, apparently, caused a rise of about 20 000 sec-ft. in the river. The second flow in Dry Creek if assumed as equal to the first, and added to the Arkansas River at the Denver and Rio Grande Section, will give 86 100 + 20 000 = 106 100 sec-ft. as the peak flow at Pueblo. (In this connection, it is interesting to note that the figures for Dry Creek indicate a run-off of nearly 300 sec-ft. per sq. mile, the drainage area of the creek being about 70 sq. miles.)

From the United States Topographic Sheets are obtained the following drainage areas tributary to the Arkansas River between Canon City and Pueblo:

South Side:

Pueblo to Boggs Creek	20	sq. miles
Boggs "	25	" "
Rock "	67	" "
Peck "	46	" "
Rush "	34	" "
Red "	42	" "
Ritchie "	40	" "
Hardscrabble Creek	186	" "
Coal Creek	28	" "
Oak "	72	" "
Chandler Creek	38	" "
Total	598	" "

North Side:

Dry Creek	71	" "
Turkey "	215	" "
Beaver "	260	" "
Eight-Mile Creek	140	" "
Oil Creek	456	" "
Total	1 142	" "
Fountain Creek	930	" "

Although a drainage area of about 3 000 sq. miles is tributary to the Arkansas above Canon City, it is known that this area did not contribute, appreciably, to the flood, as the automatic gauge on the river at Canon City recorded a maximum flow of about 4 000 sec-ft. It is also known of the areas previously tabulated, that only a few flooded excessively. Fountain Creek is excluded from the discussion for the present, since it joins the Arkansas River below the Pueblo Station.

Water from the major part of the Turkey Creek drainage was held up by the Teller Reservoir, and from the Beaver drainage by the Schaeffer Reservoir, up to the time of its failure about 10 A. M., on Sunday, June 5th, 1921.

Mr. Wells, Superintendent of the C. F. and I. Company's Arkansas Valley Conduit, who is intimately familiar with the south side of the river and in touch with conditions both through personal observation and reports from his ditch riders, says:

"The flood of June 4th was caused by rain falling east of Rush Creek, which enters the river at Swallows. Very little water came down Rush Creek and there was no water either in Ritchie Gulch or Red Creek. Peck Creek, Rock Creek, and Boggs Creek were the creeks that caused the flood."

This statement is corroborated by the postmasters at Siloam and Wetmore, who state that, although a heavy rain fell in their section, it fell steadily and penetrated into the soil.

The combined drainage areas tributary to the Arkansas River, between Pueblo and Swallows on the south side, aggregate about 150 sq. miles and,

including Dry Creek on the north, 220 sq. miles. Even if the entire south-side drainage, as far west as Portland, is included, there are only 336 sq. miles (eliminating Turkey Creek and Beaver Creek). If a run-off equal to that of Dry Creek on June 2d, 1921, namely, 300 sec-ft. per sq. mile, is assumed, there would be a combined flow of 100 800 sec-ft.

A rainfall of 12 in. in 24 hours on an area of 1 sq. mile, with complete run-off, is equivalent to 640 acre-ft., or a rate of 320 sec-ft., which is nearly what has been already assumed. Such rainfall is very much higher than any reported during the storm which caused the flood of June 3d, 1921, but it could have occurred over a limited area in the Boggs Flat Section where the storm appears to have been the heaviest. Unfortunately, no accurate records are available from this area.

The authors suggest that protection be provided against a peak flow of 168 000 sec-ft. This is based on the run-off from a storm 40% greater than any recorded, using certain assumptions as to rainfall and run-off; 60% of 168 000 sec-ft. is 100 800 sec-ft. which, again, roughly corroborates the estimate of the peak flow.

The evidence seems to show that the peak flow amounted to approximately 100 000 sec-ft., possibly more, but more likely less, since the assumptions on which this estimate is based, are extremes purposely taken in order to be sure that the figure should be too large rather than too small.

The Fountain Creek flood reached Pueblo sometime after the peak of the Arkansas flood had passed, and its peak was reached at, perhaps, 3.30 A. M., Saturday, June 4th, 1921. Preliminary estimate placed this peak at 50 000 sec-ft.

A cross-section opposite the Denver and Rio Grande Mile Post 117, approximately 2 miles above Pueblo, surveyed by its engineers, gives between high-water marks the following: Width = 921 ft.; maximum depth = 9.4 ft.; area = 5 258 sq. ft.; perimeter = 940 ft.; hydraulic radius = 5.6; and slope = 0.0042.

The peak flow of Fountain Creek at this point, calculated by Kutter's formula, with $n = 0.035$, was 46 500 sec-ft., and with $n = 0.040$, 40 100 sec-ft. At that time, there may have been 75 000 sec-ft. in the Arkansas River. This would give a total volume of 120 000 sec-ft. as the combined discharge of the two streams.

Below Pueblo, the largest flood in a tributary stream occurred in the St. Charles River. As this flood was caused by the rain storm which brought about the Arkansas flood, and had a shorter distance to travel, it is likely that its peak reached the main river before that of the main flood. Definite information on this point may be secured later.

From a cross-section made by Messrs. H. D. Amsley and G. C. Price, the peak flow of the St. Charles is indicated to be about 50 000 sec-ft. If one-half of this flow was coincident with that of the peak in the main river, the combined flow below the St. Charles would be 145 000 sec-ft., to which Chico Creek added its volume.

A cross-section of Chico Creek, taken by Messrs. Amsley and Price, indicates that it carried a little more than 20 000 sec-ft., and the statement of Mr.

Edson, Foreman of the North Avondale Milling Company's ranch, fixes the time of the rise in Chico Creek at 4 A. M., June 4th, 1921. This would bring it at about the time of the peak in the Main River at this point, and would give a total of about 165 000 sec-ft.

The Huerfano River did not flood excessively, but, from gauge heights at the Ellis Dam, it is estimated that it contributed from 5 000 to 6 000 sec-ft. Below this point, the tributary streams were not unusually high, although they probably did furnish some excess water. If the sum of the tributaries between the Huerfano and La Junta is estimated as 10 000 sec-ft., the peak at La Junta was 160 000 sec-ft.

At the head of the Fort Lyon Canal, about 3 miles west of La Junta, a cross-section was taken by Messrs. Amsley and Price, and the flow estimated through this section as the peak flood is from 180 000 to 200 000 sec-ft.

At the head of the Amity Canal, near Prowers, certain flood-flow estimates were made by Oscar Hallbeck, Engineer for the Arkansas Valley Sugar Beet and Irrigated Land Company, as shown in Table 20.

TABLE 20.

Time.	Date.	Velocity, in feet per second.	Quantity, in second-feet.	Height, over dam, in feet.
11.00 P. M.	June 4, 1921.	....	2 000	....
12.00 P. M.	"	....	6 000	....
1.00 A. M.	June 5, 1921.	....	10 000	....
1.10 A. M.	"	....	15 000	2.5
1.30 A. M.	"	....	20 000	3.0
1.40 A. M.	"	....	28 000	4.0
2.00 A. M.	"	20	45 000	5 5
2.30 A. M.	"	20	55 000	6 0
3.00 A. M.	"	25	80 000	7.0
3.30 A. M.	"	25	110 000	7.5
4.00 A. M.	"	30	155 000	8 5
4.30 A. M.	"	30	170 000	9.0
8.00 A. M.	"	....	170 000	9.0
8.30 A. M.	"	....	155 000	8.5
9.00 A. M.	"	....	130 000	8.0
10.00 A. M.	"	....	130 000	8.0
11.00 A. M.	"	....	130 000	7 5
12.00 M.	"	....	110 000	7.0
1.00 P. M.	"	....	80 000	....
8.00 P. M.	"	....	....	....
9.00 P. M.	"	....	55 000	6.0

The figures given in Table 20 show a peak flow of 170 000 sec-ft. which continued for 3½ hours, with a gradual decline, and a total discharge of, approximately, 200 000 acre-ft. in 24 hours. It is probable that the peak would flatten out on the lower river and that the flow would continue for a longer time, as indicated in the measurements at the Amity Dam.

This effect is still further illustrated by measurements not yet available, taken at Syracuse, Kans. At this point, the river did not overflow its channel, but was confined between the abutments of the bridge, in a channel about 800 ft. wide. Mr. Knapp, State Water Commissioner of Kansas, estimates the peak flood at this point as not more than 50 000 sec-ft. (subject to verification later). Evidently, the time of passage of the flood must have been considerably longer, if these figures are correct.

Information now being gathered may show minor discrepancies in some of these figures and will doubtless throw more light on the actual area of origin of the flood, but it is believed that, in the main, the conclusions arrived at are correct within reasonable limits and may be of value in consideration of plans for future flood protection.

Although the greater part of the damage done was in and around Pueblo, it is evident that great losses occurred in the lower valley, particularly to irrigation structures, and a complete flood-protection plan should consider the lower valley as well as Pueblo.

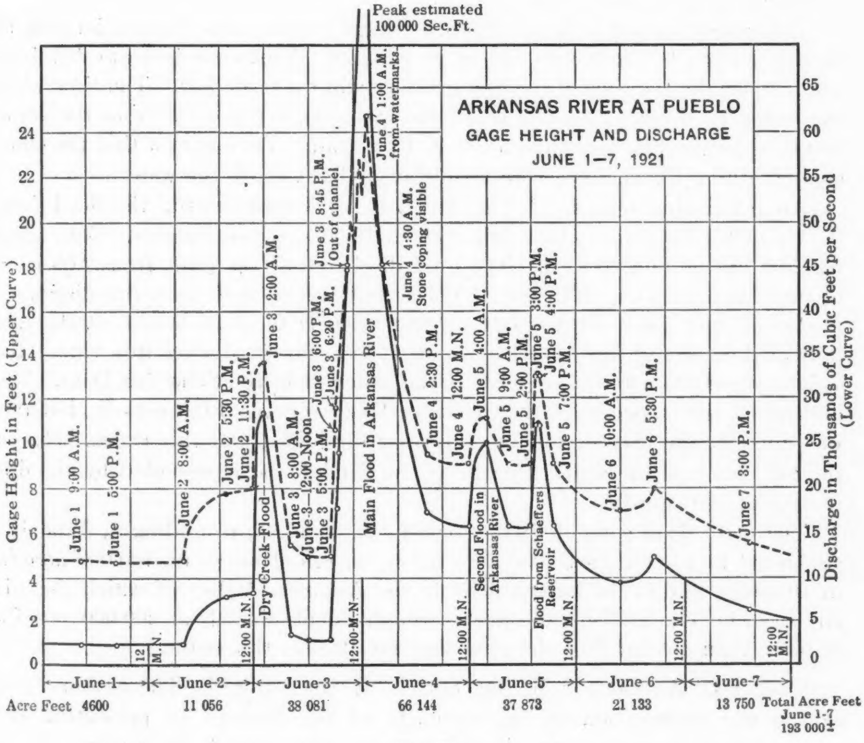

FIG. 21.

In the writer's opinion the adoption of a "conservancy district" law, similar to that of Ohio, should be passed as a first step, in order to provide the machinery for raising money for the construction of flood-prevention works. The costs of the necessary works should be assessed in proportion to the benefits to be derived, against cities, towns, corporations (public and private), and all other interests benefited.

No detailed definite study has been made as yet of the possible plans for flood protection, although various schemes have been suggested. A discussion of these plans without adequate information and reliable estimates of cost will not be attempted at this time.

It is due to Mr. Amsley's diligence and foresight in this connection that any reliable figures have been preserved. The measurements made by him June 2d, 3d, and 7th, 1921, and his subsequent work in establishing gauge heights, embrace all the actual data on record.

Fig. 21 is a hydrograph showing gauge heights and discharge of the Arkansas River, at Pueblo, from June 1st to 7th, 1921. Since the automatic gauge was lost during the evening of the flood of June 3d, 1921, no strictly accurate gauge heights are available from that time until June 6th, but approximations were obtained in various ways, from reliable statements, water-marks, inspection from a distance, counting the number of stones in the side-walls visible above water level, etc.

This graph should be quite accurate except for the time during the peak of the flood when the river was out of its channel. This time is quite definitely fixed, especially the time of the first overflow and the peak flow. The time when the water in the channel was below the stone coping is fixed from the statements of people who spent the night in City Hall. They agreed that the stone copings lining the channel were visible at daylight above the water.

An interesting feature of Fig. 21 is the peak representing the flood from the Schaeffer Reservoir which failed about 10.00 A. M., on Sunday. This water reached Pueblo at 2.00 P. M. The total discharge of the river from 2.00 P. M. to midnight, on June 5th, was 15 173 acre-ft. (computed from the diagram). Assuming that the river was falling, from 2.00 P. M. to midnight, at the rate indicated, it would have carried about 11 580 acre-ft. during this time; the difference—3 593 acre-ft.—represents the water from the Schaeffer Dam. The high-water line capacity of the Schaeffer Reservoir was 3 190 acre-ft., but this was undoubtedly exceeded before the reservoir broke, and this excess between the capacity and the water "accounted for," might be represented by the difference of 400 acre-ft.

The main flood computed from 5.00 P. M., June 3d, to midnight, June 4th, amounted to more than 90 000 acre-ft., or, in round numbers, 100 000 acre-ft. in 19 hours, and is the largest flood in the Arkansas Valley of which there is any record. The total for the week through Pueblo was about 200 000 acre-ft., or more than one-third of the total flow for 1920 at this point.

GEORGE G. ANDERSON,* M. AM. SOC. C. E. (by letter).†—The authors have earned the approbation of the members of the Society in presenting the accurate and comprehensive, though concise, statement of the features of the great flood in the Arkansas River in 1921, and the resulting situation in the City of Pueblo and the Arkansas Valley, Colorado. Such approbation is equally due to Mr. Hosea for the able and painstaking manner in which he has presented the facts regarding the flood flows of the river. It was necessary to make clear early that the great volume of the flood, and the extent of the resulting damages, came from entirely natural causes, and in the rehabilitation of the city and of the adjacent territory, serious engineering problems are presented, which call for grave consideration in their solution.

From the testimony of such observers as Mr. Wells (quoted by Mr. Hosea),

* Los Angeles, Cal.

† Received by the Secretary, October 3d, 1921.

that the greater part of the run-off, at least on the south side of the river, resulted from the rainfall east of Rush Creek, and in the absence of records of rainfall from a greater number of rain-gauge stations throughout the area covered by the general storm, it may be a warrantable assumption that one-half the volume of 100 000 acre-ft. came from less than 300 sq. miles of drainage area. From information obtained by the writer, while investigating the conditions on Beaver Creek and the upper reaches of Fountain Creek, shortly after the floods, it would appear, however, that intense rainfall occurred on the north side of the river and caused run-off which, undoubtedly, contributed to the flood in some volume.

Mr. Hosea states that the "Beaver Creek drainage was held up by the Schaeffer Reservoir until the time of its failure, Sunday, June 5th, 10 A. M." Of the total Beaver Creek drainage of 260 sq. miles, there are only 136 sq. miles above the Schaeffer Reservoir. On some considerable portion of the remainder of 124 sq. miles, there was a very heavy rainfall and a consequent large volume of run-off.

At the Schaeffer Dam a measurement, not by a rain gauge, indicated that 4 in. of rain fell during the night of Friday, June 3d, and the precipitation was stated to have been greater at Beaver Park, about 7 miles southwest. The official record at Florence shows only 0.99 in. of precipitation on June 3d, and only a slight run-off occurred in Eight-Mile Creek during that night. These facts may help to define the approximate western limit of the great storm, although a detail in one element may be pointed out in this connection, and, later, elaborated in a broader aspect of the circumstances as they are now known.

Although the total precipitation for June 3d, at Florence, as given in Table 5, is 0.99 in., it is unlikely that this amount was the total for that day of the rainfall which caused the run-off accumulating in the flood at Pueblo during the night of June 3d and the early morning of June 4th. It is much more likely that the quantity given as the total for that day at Florence was the rainfall up to 6.00 P. M., which hour is the beginning and ending of the Weather Service Bureau's day at various stations in Colorado, for instance, as at Colorado Springs and Lake Moraine, while, at Victor, the day begins and ends at 4.00 P. M. It is noticeable that on the following day, Saturday, June 4th, at Florence the total rainfall is the maximum quantity, 3.31 in. It is probably correct to state that the greater part of this fell during the evening, after 6.00 P. M., and the night of June 3d, and in all probability thus contributed some volume to the flood at Pueblo and, also, to its peak flow.

Although only a slight run-off was observable in Eight-Mile Creek on the night of June 3d, Brush Hollow Creek, the next creek eastward, discharged a very high run-off. Subsequent measurements indicated more than 6 000 cu. ft. per sec. as the peak flow from a drainage area not in excess of 25 sq. miles. The next creek eastward is Beaver Creek which is known to have increased in discharge below the Schaeffer Dam before that structure failed, but all evidences of the volume on that night have been obliterated.

The next creek eastward is Turkey Creek. Although it may be correct to state that the Turkey Creek Reservoir retained the stream flow which occurred

above it, there were evidences of quite heavy rainfall on that area, with considerable damage to roads and irrigation ditches. If that heavy rainfall did not extend into the area which the Turkey Creek Reservoir does not intercept, there is indication that the intensity of the storm varied in different localities, which is quite probable. For instance, on the Dry Creek area, the next eastward to Turkey Creek, there is every evidence of intense rainfall and a large run-off.

From the foregoing comments, the writer is of the opinion that, although the greater volume of the Arkansas River flood came from the south side of that stream, and largely east of Hardscrabble Creek, considerable volumes were added from streams on the north side, and that the tributary area of these streams cannot be wholly disregarded in these considerations.

At the Schaeffer Reservoir the heavy rainfall did not commence until about 7.30 P. M., on June 3d, and the consequent run-off may not have reached Pueblo at the time of peak flow, but, undoubtedly, it did add something to the total volume. The flow of Beaver Creek at and below the Schaeffer Dam did not exceed 90 cu. ft. per sec. until 4.00 A. M., on June 4th, when the water surface of the reservoir reached the spillway level.

The writer is of opinion that the statement, "in the two largest storms [of the Arkansas Valley], namely, those of May, 1894, and June, 1921, the average rainfall increases quite uniformly with the elevation of the drainage area", is apt to be misleading, and to require revision in its application to the storm of June, 1921.

As has been stated, the rainfall at the Schaeffer Dam, at an elevation of 5 700 ft., on Beaver Creek, during the night of Friday, June 3d, was about 4 in. The rainfall, at Victor (elevation, 9 775 ft.), from June 3d, 4.00 P. M., to June 4th, 4.00 P. M., was 2.08 in. Victor is on the western slope of Pike's Peak, while Lake Moraine, at an elevation of 10 200 ft., and Colorado Springs, at an elevation of 6 500 ft., are on the eastern slope and in the Fountain Creek drainage. At the two latter points, the precipitation is given, as recorded in Tables 4 and 5, from 6.00 P. M. of one day to 6.00 P. M. of the next. Table 21 shows the comparative rainfall, in inches, at these points.

TABLE 21.

Date.	Victor.	Lake Moraine.	Colorado Springs.
June 3, 1921	0.03	0.65	5.00
June 4, "	2.08	3.68	4.40
June 5, "	1.55	1.40	1.26
June 6, "	0.37	0.18	0.42
June 7, "	0.01	0.00	0.01
Total	4.04	5.91	11.09

At these three stations, the total rainfall for five days shows that the lower elevation actually had more than twice as much precipitation as the average of the higher elevations. By analyzing the daily quantities, keeping in mind the different hour to which the report refers, the record shows that prior to 4.00 P. M., on June 3d, 0.03 in. of rain fell at Victor, prior to 6.00 P. M.,

0.65 in. fell at Lake Moraine, and prior to 6.00 P. M., 5.00 in. fell at Colorado Springs.

The detailed record at Colorado Springs is much more illuminating as to the character of the storm, and, between rainfall and altitude, to the relation for this particular storm:

	Rainfall, in inches.
June 3d, 1921, 3.30 P. M. to 6 P. M.	5.00
June 3d, " 6 P. M. to June 4th, 2 A. M.	4.20
June 4th, " 2 A. M. to 6 P. M.	0.20
June 4th, " 6 P. M. to June 5th, 6 P. M.	1.26
June 5th, " 6 P. M. to June 6th, 6 P. M.	0.42
June 6th, " 6 P. M. to June 7th, 6 P. M.	0.01
Total	11.09

The total rainfall of 9.2 in. at Colorado Springs from 3.30 P. M., June 3d, to 2.00 A. M., June 4th, is comparable with the rainfall of 2.08 in. reported at Victor for June 4th, which really occurred after 4.00 P. M., on June 3d, and probably continued, as at Colorado Springs, until the early morning of June 4th. A similar comparison applies to Lake Moraine, with the alteration that the daily periods are parallel as previously given.

These three Weather Bureau Stations are fairly comparable, for, apart from being the only stations in the path of that particular storm, they are situated, relatively, in the general line followed by the storm of June 3d, which apparently was almost directly at right angles to the front line of the mountains, with a northeast tendency. Generally speaking, this is a characteristic of these summer storms on the Colorado Plateau.

This actual reversal of rainfall increasing with altitude is not unusual in storms of this character, locally and colloquially called "cloudbursts", since this storm which caused the Pueblo flood, was on an unusual and exaggerated scale. As shown in Table 5, it is traceable, with varying intensity of rainfall, along the eastern slope of the Rockies, almost entirely throughout Colorado, with some favored localities excepted, as, for instance, Trinidad, and well into New Mexico, as far south as Albuquerque, on that night of June 3d.

Such storms are usual during the summer months, and although they originate within the mountains, ordinarily at not excessive elevations, and cause some precipitation inside the front range, the greater part of the rainfall, as in this case, ordinarily occurs on the high plateaus or plains areas. There are those who declare that "cloudbursts" do not occur above an elevation of 9 000 ft.

Under such conditions, it does not seem to the writer that any sound deductions can be made from "average" rainfalls, say, at elevations of from 6 000 to 12 000 ft. For the purpose of establishing merely the relation between rainfall and elevation, it would seem that the method used in Table 6 and the averages derived therefrom are misleading. Considering the column for June, 1921, it is noticeable that the fourth group, 6 000 to 12 000 ft., includes Colorado Springs, Calhan, and Monument, all of which are stations outside

the mountains proper and on the plateau or plains area. This group includes Cuchara Camps, with a rainfall of only 1.10 in. Cuchara Camps is in the mountains, at an elevation of about 9 000 ft., and near the head-waters of the Huerfano, but outside the belt of this particular storm. The Huerfano contributed only a small volume to the flood-waters of the Arkansas River, east of and below Pueblo.

The average rainfall was 6.29 in. at the typical plateau stations in that fourth group, Colorado Springs, Monument, and Calhan, and for the typical mountain stations, Victor and Lake Moraine. These stations are close to one another and were in the general path of the storm. The first group of three lie outside the mountains, and the second group well inside of the mountain area. Comparing the plateau stations, there is:

	Elevation.	Rainfall.
Colorado Springs	6 000 ft.	10.66 in.
Monument	7 000 ft.	3.72 "
Calhan	6 500 ft.	4.48 "

In the third group, if Trinidad, plainly outside the belt of the intense storm, with a rainfall of 1.05 in. is eliminated, the average rainfall at Canon City and Florence is 5.08 in., which is slightly more than that of Victor and Lake Moraine, 4 000 ft. higher.

In the second group, at an elevation of from 4 000 to 5 000 ft., Pueblo with 4.21 in. of rainfall in 72 hours, is the only station which should properly be compared. Ordway, Two Buttes, and Rocky Ford were certainly outside the limit of the storm.

In the first group the stations, Lamar and Los Animas, were also outside the storm belt.

Another set of four groups could thus be set up, as follows:

	Elevation.	Rainfall.
Pueblo	4 000 to 5 000 ft.	4.21 in.
Canon City, Florence............	5 000 to 6 000 ft.	5.08 "
Colorado Springs, Calhan, Monument	6 000 to 7 000 ft.	6.29 "
Victor, Lake Moraine.............	10 000 ft.	4.86 "
Average....................................		6.81 in.

The probable relation of rainfall to elevation in such storms is thus more clearly indicated, and the fact is revealed that, at the lowest elevation, Pueblo, the rainfall was only slightly less than at almost the summit of the drainage area. The much more significant feature is that directly within the path of the storm, the average rainfall from various typical stations from which conclusions might be made, just as fairly if not more accurately, by way of forecast of the results in any recurrence, is more than twice that which the authors find and on which they base some important conclusions.

The official record of precipitation at Colorado Springs shows the maximum of any station within the storm area, and is the maximum rainfall recorded at any station in the period of 30 years. It is possible and probable indeed that

rainfall, as intense as the 9.2-in. rain, which fell at Colorado Springs during the 10½ hours, from 3.30 P. M., June 3d, to 2.00 A. M., June 4th, occurred west of Pueblo, in some considerable portion of the Arkansas River area, which district is assumed to have yielded the greater part of its volume to the flood. The absence of a sufficient number of well placed rain-gauge stations within the areas subject to these torrential summer storms has never been so well illustrated as on this occasion. If records had been available from a large number of such stations during this particular storm, much more really satisfactory deductions could have been made in place of the estimates, more or less dependable, than can now be arrived at, on the most conservative basis.

The consequent defects are apparent in the tabulation of assumed percentages of run-off, which to the writer, appear to be unreliable largely because they again are based on the difference in elevation, rather than on the rainfall that did occur at each elevation, and the conditions affecting run-off that prevailed at the time of the storm. It would seem, for instance, that a rainfall of 9.2 in. in 10½ hours at Colorado Springs, more than 5 in. of which occurred in 2½ hours, would have resulted in a greater percentage of run-off than from 3.68 in. at Lake Moraine in the same time. From personal observation in many such "cloudbursts"—not of the extreme intensity of this particular storm—it has frequently appeared to the writer that the run-off was almost complete and immediate. It would also appear that, above an elevation of 7 000 ft., with less rainfall within relatively the same period, the run-off would be less than at the lower elevations. On this occasion, in the region of Victor and Lake Moraine, the precipitation above an elevation of about 10 500 ft. was in the form of snow, and, in one instance at that elevation, the run-off from about 10 sq. miles of drainage area did not exceed 50 acre-ft. per day for 3 days after the storm, while the average precipitation, as previously stated, was 4.86 in.

It may be that because "probably one-half this volume [100 000 acre-ft., which passed Pueblo] came from less than 300 sq. miles of drainage area between Hardscrabble Creek and Pueblo, * * * the storm which caused the flood was far from a maximum". It was a maximum, so far as flood volume of the Arkansas River passing Pueblo during more than 30 years is concerned, and so far as intensity of rainfall in adjacent territory, as at Colorado Springs, is concerned. A greater rainfall was recorded at Canon City during the storm of 1894—5.06 in. as compared with 3.40 in. in 1921. Unfortunately, that is the only station in the Arkansas Valley above Pueblo, at which comparison may be made.

Assuming the accuracy of the judgment that 50 000 acre-ft. came from 300 sq. miles, an equally intense rainfall with an equally great percentage of run-off from 1 000 sq. miles would result in a flood of more than 166 000 acre-ft., not three times the volume of the recent flood. It is conceivable, however, that a rainfall of an intensity equal to that at Colorado Springs (elevation 6 000 ft., 9.2 in. in 10½ hours) could occur over all the drainage area in the Arkansas Valley above Pueblo and below Canon City, all of it below an elevation of 6 000 ft., and from 1 740 sq. miles, in place of 1 000 sq. miles, produce a flood

equal to or greater in relative volume than that yielded from 300 sq. miles on June 3d, which would approach a volume "three times that of the recent flood".

Under such conditions, with a total run-off of more than 300 000 acre-ft., it may be reasonable to expect a peak flow of 168 000 sec-ft. in the Arkansas River at Pueblo, and it may be essential to provide for that volume, since it is only 68% in excess of the recent flood, although that is the maximum discharge of record in a period of more than 30 years, following a precipitation which is also the maximum in the same period, with the single exception of the record at Canon City.

The results anticipated from the Fountain Creek drainage area, following on a similar study, are not equally convincing, however. The flood of June 3d in Fountain Creek, at Pueblo, showed a total volume of 50 000 acre-ft. and a peak flow of 50 000 sec-ft., both of which are, apparently, the maximum of which there is any record.

From the whole drainage area of 930 sq. miles, the total discharge of 50 000 acre-ft. is equivalent to an average run-off of practically 1 in. Approximately, one-half of that area is below an elevation of 6 000 ft., and the greatest rainfall occurred at Colorado Springs, practically at that elevation. Therefore, on the basis of the authors' tabulation of assumed percentage of run-off—55%—the volume of the flood would have been due to an average rainfall of 1.8 in. There are four rain-gauge stations within the Fountain Creek area, Lake Moraine, Monument, Colorado Springs, and Pueblo, and the rainfall at these stations, on the night of June 3d-4th, as nearly as it may be established from the reports, was:

Lake Moraine	3.68 in.
Monument	2.90 "
Colorado Springs	9.20 "
Pueblo	3.09 "
Average	4.72 in.

In order to produce a flood of 164 000 acre-ft., on the basis of a run-off of 55%, the average rainfall would have to be in excess of 6 in. Although such anticipated flood might be possible, it does not seem to be probable, in view of the facts that the recent flood in Fountain Creek at Pueblo was a maximum alike in total volume and peak flow, as was the rainfall at Colorado Springs and other stations, with the single exception of Lake Moraine. Such anticipation, at any rate, cannot very well be based on the related data in the recent experience.

As an incidental item in connection with these estimates or forecasts, it may be noted that, on the same basis, the area of approximately 183 sq. miles between the site of the suggested detention reservoir on Rock Creek and Pueblo, might produce a flood greater in peak flow than the capacity of the channel within the levees, which existed in Pueblo prior to June 3d, 1921.

It may be proper, and permissible, to bear testimony to the accuracy of some of the detailed statements made by the authors and by Mr. Hosea. At the

time of original construction in 1910, the capacity of the Schaeffer Reservoir, at the spillway level, was approximately, 3 190 acre-ft. Some silting had occurred in the basin, but reduction from that cause was offset by the storage above the spillway level which had occurred prior to the failure. For 12 hours or more preceding the failure on Sunday morning, June 5th, the discharge of Beaver Creek had ranged from 1 500 to 4 000 sec-ft., or more.

The writer passed through the Lower Arkansas Valley, below La Junta, on the morning of June 4th, finding the contributions to the river flow from tributary streams generally as presented. There was some flood flow in Timpas Creek, immediately west of La Junta, estimated at about 1 000 sec-ft., and that was probably all diverted before its junction with the Arkansas River.

In their consideration of "Reconstruction and Flood Control", the authors, very properly, have not attempted to do more than give a general outline of possible alternative and combined methods of improvement that would prevent similar damage in the future, and only in such general terms will comments thereon be submitted.

It would seem to be inevitable, and it certainly would be desirable, to combine any reconstruction work in the City of Pueblo with necessary plans for some improvement of the conditions along the Arkansas River below Pueblo, where very great damage was sustained by irrigation works. The interests of the city and the adjacent farming district are so interdependent that some plan incorporating improvement of mutual benefit should be devised, if at all possible. The authors indicate the necessity of some such co-operative plan, indeed, in the remark that

"* * * the enlargement of the channel through Pueblo in order to carry the peak flow of the river would not benefit property interests in the valley below; in fact, it might have the effect of slightly increasing the flood peaks in the lower river."

In view of this statement, and for other reasons readily apparent, the suggestion made by Mr. Hosea that the creation of a conservancy district, as the first step toward the rectification of existing damages and the prevention of a recurrence in the future, is probably the best course to be pursued in consideration of all the interests affected. It is also probable that some combined method of flood detention storage and channel enlargement will prove the only effective means.

The location of a storage reservoir or reservoirs of adequate capacity, would be confined, undoubtedly, to the channel of the streams. It is improbable that any site of sufficient capacity can be found at reasonable cost on tributary streams, while experience in the recent flood indicates without doubt that the great volume of the flood occurs outside of the front range, or foot-hills of the mountains. As pointed out by the authors, the only known available site, on the Arkansas River at Rock Creek Canyon, has distinct limitations, and these limitations are not entirely confined to the feature of the development of adequate capacity. Although the authors' studies show a requirement of 210 000 acre-ft. capacity, it may be concluded that 100 000 acre-ft. is the economic limit, and even that is curtailed by practical considerations which seriously affect its possible advantages.

Two important railroads, the Denver and Rio Grande and the Santa Fe, are located in what would become the basin of the reservoir. Their removal to other locations is inevitable, and may occasion considerable physical difficulty. It is not conceivable that the railroad companies would consent to leave the tracks where they are, although the authors suggest that in the remarks that "only the most unusual floods would submerge the railroads * * * by storing water in the reservoir" and "the effect of submergence in comparatively quiet water might cause less damage than the high velocities under present conditions". It may be definitely concluded that practical railroad managers would not consent to assume such chances.

It may be anticipated, also, that the residents in the lower sections of the City of Pueblo and the irrigation interests in the Lower Arkansas Valley will not readily accept a proposal to construct, by a high dam in the channel of the stream, a reservoir of large capacity in such location as would seem to imply a continuing menace to life and property. Such an attitude would merely be the reaction from recent experiences. When it was overcome another attitude, probably as important, would develop in the probable desire of the irrigation interests to utilize the reservoirs for irrigation purposes for "hold-over storage". The authors have touched briefly on this in their concluding remarks, in which it is stated that "it is evident that the supply for additional irrigation development is very limited". Thorough investigation of the stream flow of the Arkansas River and appropriations for storage supply would show, it may be surmised, that the available supply is now well nigh exhausted by "live rights" attached to projects of capacity, except under such abnormal conditions as those which prevailed on June 3d-4th. Any effort to develop "hold-over storage" in a flood-detention reservoir would probably further complicate the distribution of the stream flow among the present water consumers. Even though it may be true that the available flow is now exhausted by the claims of such prior appropriators, there is also the possibility that the water impounded in the detention reservoir, under the "hold-over storage" plea, might seriously lessen its capacity for its main purpose of flood detention.

The authors give no encouragement to the proposition that such a detention reservoir be used as storage for part of the domestic supply for the City of Pueblo, for the reason that the basin would soon fill with silt. To that perhaps could be added the inadvisability of depending on Arkansas River water, as a source of water supply, under all conditions, if in no other respect than that of purity.

The greatest reliance for future protection in Pueblo will undoubtedly be placed on channels of adequate capacity throughout the city. It would also be desirable to raise street and other grades as much as possible. The most superficial study of the street profile in relation to the top of the levees along the river and of the contours in the flooded district, is sufficient to indicate that, if life and property is to be reasonably free from undue hazard, in addition to ample channel capacity in the future, there is entailed a distinct elevation of street and property grades, with all that these changes will involve.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

RAINFALL AND RUN-OFF STUDIES

Discussion*

BY MESSRS. DANA M. WOOD, C. F. MARVIN, RUDOLPH HERING, AND OLIN H. LANDRETH.

DANA M. WOOD,† M. AM. SOC. C. E. (by letter).‡—Engineers are still divided in their opinion regarding the advisability of publishing rainfall and run-off statistics for the calendar year or for some universally adopted climatic year. A committee of the Boston Society of Civil Engineers (the Run-Off Committee) sent out questionnaires to many engineers, asking their opinions on this question, and the replies were divided about equally. Many seemed to feel that particular studies in a specific locality might require the use of the climatic year, but opinion differed as to the best common climatic year to use, varying as it does in different parts of the country.

The advantage in adhering to the calendar year for rainfall and run-off statistics appears to be threefold: (1) because of the non-uniformity of the climatic year; (2) it is the method used by the U. S. Weather Bureau in publishing the mass of rainfall figures already available, and the difficulties in bringing about a change, including a revision of past publications, would seem to be insurmountable for the present at least; and (3) the business reports of most municipalities and companies are on a calendar-year basis, and these reports often contain statistics of the nature discussed. The argument may sound like that of "locking the barn door after the horse is stolen" but, nevertheless, it is real. It is difficult to change long-established practice, even to substitute improved methods.

In regard to run-off records, engineers seem to be very definitely committed to the climatic year beginning October 1st, by the adoption of this system by the Water Resources Branch of the U. S. Geological Survey. The chief argument in favor of its general adoption seems to be that it permits of the continuous uninterrupted investigation of winter-flow conditions in those localities subject to ice conditions, and of the office preparation of report

* This discussion (of the paper by C. E. Grunsky, M. Am. Soc. C. E., published in September, 1921, *Proceedings*, and presented at the meeting of October 5th, 1921), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Boston, Mass.

‡ Received by the Secretary, October 5th, 1921.

data at times of the year when field work is not so necessary; even this is not strictly true for all localities. The argument that it permits the issuance of published reports six months earlier than would otherwise be possible, does not seem to have so much force in recent years.

Summing up, it appears to matter little to the individual engineer which system is used because, in the majority of cases, he is forced to make his own transpositions and calculations on the basis of what in his judgment is the best division of the year for the locality he is studying. As long as the diversity of opinion exists, every one cannot be entirely satisfied, and the only satisfactory solution would be to use different divisions of the year for different sections of the country, and this also has its objections.

Referring to the discussion of "Rainfall in the Climatic Year",* the question arises as to what is a normal rainfall. According to Weather Bureau data, the annual normal is the sum of the normals for all the months, and the normal for each month is the average for the period of record. Furthermore, normals are computed at stipulated intervals and are not always changed between times to obtain the correct figure to date.

Attention has been called to the value of the median.† In Mr. Grunsky's paper, the study of frequency might be made theoretically more correct by the use of the median, but it is admitted that the final conclusions would be practically the same in this case, because there is so little departure between the two for the records used. Fig. 5 indicates this; but it is often not the case in other localities.

At the risk of repeating Mr. Grunsky's arguments, emphasis should be laid on the fact that, regardless of any arbitrary division of the year, the precipitation in the few months previous to the new year has a marked effect on ground-water levels and, therefore, on run-off. In other words, any method of analysis depending on the determination of the ratio of run-off to rainfall in any one given month will lead to wrong conclusions. Such ratios are of value for seasonal variations, but the length and dates of the several seasons will vary from year to year.

A fairly reliable estimate of the probable average, minimum, and maximum annual rainfall can be made for a given water-shed by the methods given in Mr. Grunsky's paper. Other excellent papers indicate somewhat different methods for the same purpose.

In making estimates of the power available in a given water-shed, most engineers now desire both a hydrograph and a duration-of-flow curve, the latter including both the extreme maximum and minimum flows of record. If the annual run-off for the three typical years can be determined from rainfall statistics, the question of its relative distribution throughout the year is still to be determined.

So many different factors affect run-off that even with the same rainfall, in two different years, the run-off may be quite different both in quantity and distribution. The condition of the ground-water storage is an important factor, as well as those pointed out in the paper.

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* *Proceedings,* Am. Soc. C. E., September, 1921, p. 213.

† *Engineering News-Record,* Vol. 80, p. 628.

In order to determine the probabilities regarding run-off, some information which covers actual stream-flow measurements, even if for a short term of years, is imperative.

For a number of years the writer has intermittently been collecting and comparing run-off records compiled by a method similar to that used by Mr. Grunsky for rainfall records.

Duration curves of flow for the three types of years mentioned furnish a starting point. Discussing the average duration curve only in that which follows, the basis for comparison is always taken as the time scale, using it as percentage of total time for the period covered by the records and comparing flows for every 5% of the time. The flow is expressed as a percentage or ratio of the normal flow, the normal flow being taken as either the average or the median as may best suit the investigator's ideas on that subject.*

The value in using such a method lies in the comparisons afforded between what to the casual observer are entirely unlike streams. Records can be compared for widely different sizes of water-shed and for widely different annual run-offs; it is the relative distribution which is being studied.

TABLE 10.—COMPARISON OF MERRIMACK AND CONNECTICUT RIVER RECORDS.

Percentage of time.	Ratio of Flow for Each Given Percentage of Time to Average.			
	October, 1880, to September, 1899.		October, 1900, to September, 1915.	
	Merrimack River at Lawrence, Mass. (4 570 sq. miles).	Connecticut River at Holyoke, Mass. (8 390 sq. miles).	Merrimack River at Lawrence, Mass. (4 452 sq. miles).	Connecticut River at Orford, N. H. (3 100 sq. miles).
100	0.005*	0.00*	0.008*	0.054
95	0.21	0.21	0.145	0.17
90	0.28	0.25	0.25	0.21
85	0.32	0.29	0.31	0.24
80	0.35	0.32	0.36	0.28
75	0.38	0.36	0.40	0.32
70	0.42	0.40	0.44	0.36
65	0.48	0.45	0.47	0.40
60	0.52	0.51	0.51	0.45
55	0.58	0.58	0.56	0.50
50	0.68	0.64	0.63	0.56
45	0.73	0.72	0.70	0.63
40	0.84	0.81	0.78	0.72
35	0.96	0.94	0.88	0.82
30	1.12	1.08	1.02	0.96
25	1.30	1.24	1.19	1.15
20	1.52	1.47	1.46	1.38
15	1.82	1.77	1.82	1.81
10	2.22	2.24	2.34	2.42
5	3.03	3.06	3.14	3.62
0	11.28	9.40	10.06	10.54
Average, cubic feet per second per square mile..	1.56	1.46	1.333	1.712

*Flow controlled by power plant.

This naturally brings out the possibilities of classing streams as to their drainage-basin type, with or without varying degrees of storage development, etc. Of course, there are many gradual gradations from one class to another.

* *Stone and Webster Journal*, February, 1917.

This is true, also, for the classifications given in Mr. Grunsky's paper, as it is necessary sometimes to adopt a value between those given.

In accordance with the writer's method, comparisons should be, as in rainfall studies, for the same period of years, but short-term records can be extended to long-term records just as readily as with rainfall records.

Studies along these lines have not been made to a sufficient extent to determine the full possibilities of the method and a classification system; but as an indication, Tables 10, 11, and 12 are given, covering several records analyzed in the foregoing manner. With such variations in actual stream-flow records in a restricted locality, as shown by these tables, the dangers of relying entirely on rainfall records are obvious.

TABLE 11.—PERCENTAGE OF TIME-DISTRIBUTION OF FLOW TABLE, MAINE STREAMS, 1913-20 (8 YEARS).

Percentage of time available.	RATIO OF INDIVIDUAL TO AVERAGE FLOW.							
	St. John River at Van Buren. av. 1.67. Drainage area = 8 270 sq. miles.	Machias River at Whitneyville. Av. 2.41. Drainage area = 465 sq. miles.	Union River, West Branch, at Amherst. Av. 2.18. Drainage area = 140 sq. miles.	Penobscot River at West Enfield. Av. 1.91. Drainage area = 6 600 sq. miles.	Piscataquis River near Foxcroft. Av. 2.65. Drainage area = 286 sq. miles.	Kennebec River at the Forks. Av. 1.76. Drainage area = 1 570 sq. miles.	Kennebec River at Waterville. Av. 1.86. Drainage area = 4 270 sq. miles.	Presumpscot River at Sebago Lake. Av. 1.50. Drainage area = 436 sq. miles.
100	0.06	0.009	0.04	0.12	†	0.08	0 013	†
95	0.13	0.18	0.10	0.32	0.06	0.31	0.25	0.31
90	0.16	0.23	0.14	0.37	0.09	0.39	0.34	0.40
85	0.19	0.28	0.19	0.40	0.12	0.45	0.40	0.53
80	0.22	0.31	0.22	0.42	0.14	0.52	0.44	0.69
75	0.26	0.35	0.27	0.45	0.19	0.58	0.47	0.81
70	0.31	0.39	0.32	0.47	0.23	0.64	0.50	0.86
65	0.36	0.45	0.38	0.51	0.28	0.69	0.52	0.92
60	0.41	0.51	0.46	0.54	0.33	0.75	0.54	0.96
55	0.46	0.57	0.54	0.58	0.41	0.82	0.56	0.99
50	0.52	0.63	0.62	0.63	0.50	0.88	0.58	1.02
45	0.59	0.70	0.72	0.70	0.58	0.93	0.62	1.04
40	0.67	0.79	0.84	0.77	0.68	0.99	0.67	1.06
35	0.76	0.94	0.97	0.88	0.77	1.06	0.77	1.09
30	0.91	1.23	1.11	1.00	0.85	1.13	0.92	1.13
25	1.10	1.32	1.33	1.15	1.03	1.20	1.14	1.18
20	1.39	1.56	1.58	1.39	1.37	1.27	1.44	1.21
15	1.85	1.85	1.93	1.77	1.86	1.38	1.81	1.24
10	2.68	2.32	2.44	2.24	2.62	1.56	2.32	1.28
5	4.03	2.98	3.33	3.15	3.84	2.24	3.22	1.36
0	8.76*	9.91*	6.65	7.04*	26.1	8.59	11.1	21.05

* Peak discharge.

† Controlled by plant above.

Table 12 does not furnish an absolute comparison between records because the time-period is variable. It does show how several types of streams vary in run-off distribution. Extreme cases are selected.

The lengths of the records are as follows:

Column (2), 1878-1915 = 38 years, average flow = 0.538 cu. ft. per sec. per sq. mile.

Column (3), 1912-1916 = 5 years, average flow = 1.724 cu. ft. per sec. per sq. mile.

Column (4), October, 1905-August, 1917 = 12 years, average flow = 1.63 cu. ft. per sec. per sq. mile.

Column (5), October, 1903-September 1920 = 17 years, average flow = 2.19 cu. ft. per sec. per sq. mile.

Column (6), October, 1912-September, 1920 = 8 years, average flow = 1.61 cu. ft. per sec. per sq. mile.

Column (7), October, 1904-September, 1920 = 16 years, average flow = 1.68 cu. ft. per sec. per sq. mile.

Column (8), October, 1880-September, 1920 = 40 years, average flow = 1.51 cu. ft. per sec. per sq. mile.

Column (9), October, 1905-September, 1920 = 15 years, average flow = 1.63 cu. ft. per sec. per sq. mile.

Column (10), October, 1887-September, 1920 = 33 years, average flow = 1.50 cu. ft. per sec. per sq. mile.

TABLE 12.—PERCENTAGE OF TIME-DISTRIBUTION OF FLOW TABLE, MISCELLANEOUS STREAMS, VARYING PERIODS.

Percentage of time available.	RATIO OF INDIVIDUAL TO AVERAGE FLOW.								
	Mississippi River at at Keokuk, Iowa. Drainage area = 119 000 sq. miles.	Slippery Rock Creek at Wurtemburg, Pa. Drainage area = 400 sq. miles.	Kennebec River at Bingham, Me. Drainage area = 2 660 sq. miles.	Pemigewasset River at Plymouth, N. H. Drainage area = 615 sq. miles.	Housatonic River at Falls Village, Conn. Drainage area = 644 sq. miles	Connecticut River at Sunderland, Mass. Drainage area = 8 000 sq. miles.	Merrimack River at Lawrence, Mass. Drainage area = 4 452 sq. miles.	Androscoggin River at Errol Dam, Me. Drainage area = 1 095 sq. miles.	Presumpscot River at Sebago Lake, Me. Drainage area = 436 sq. miles.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)
100	0.156	0.029	0.134	0.027	*	0.052*	*	0.045*	0.018*
95	0.28	0.042	0.25	0.16	0.135	0.17	0.19	0.44	0.24
90	0.32	0.063	0.30	0.19	0.18	0.21	0.27	0.52	0.41
85	0.36	0.088	0.35	0.22	0.22	0.25	0.32	0.58	0.53
80	0·40	0.11	0.40	0.25	0.28	0.29	0.36	0.63	0.65
75	0.44	0.15	0.44	0.27	0.33	0.32	0.39	0.67	0.77
70	0.49	0.19	0.48	0.30	0.37	0.36	0.42	0.72	0.82
65	0.54	0.26	0.52	0.33	0.41	0.41	0.45	0.77	0.86
60	0.61	0.33	0.56	0.365	0.47	0.46	0.51	0.81	0.93
55	0.67	0.42	0.62	0.40	0.55	0.52	0.57	0.86	0.98
50	0.75	0.48	0.67	0.45	0.62	0.58	0.63	0.90	1.02
45	0.83	0.53	0.73	0.52	0.74	0.67	0.71	0.94	1.04
40	0.94	0.64	0.78	0.61	0.84	0.77	0.80	0.98	1.08
35	1.05	0.75	0.86	0.72	0.97	0.89	0.92	1.02	1.12
30	1.18	0.90	0.97	0.88	1.13	1.05	1.13	1.06	1.16
25	1.35	1.08	1.10	1.09	1.29	1.27	1.25	1.10	1.21
20	1.55	1.64	1.33	1.44	1.50	1·54	1.49	1.16	1.26
15	1.78	1.76	1.67	1.90	1.88	1.90	1.79	1.25	1.34
10	2.05	2.31	2.22	2.52	2.40	2.44	2.22	1.46	1.43
5	2.49	3.52	3.10	3.91	3.07	3.28	3.00	2.22	1.55
0	4.9	25.8	8.7	13.9	8.5	8.0	11.9	7.0	21.0

* Controlled flow.

To study the monthly variations, it has been the writer's custom to prepare the duration-of-flow table so that duration-of-flow curves for each month can be also prepared. Obviously, the median for each month may be utilized in studying normal seasonal variations, and the duration curves indicate monthly extremes.

This general method can and has been used for obtaining a flow-duration curve where the run-off records are meager. Working from the rainfall to the estimated run-off for a given type of year, the distribution of this annual can be assumed in accordance with that in neighboring or similar basins for which there are reliable run-off records. If short-term records are available on the stream under investigation, the relative distribution for the period covered can be compared with that of other streams for the same period, and a neighboring record selected showing similar characteristics. From these data, an estimated curve for the longer period of years can be obtained, as described for the rainfall records.

Maximum and minimum probable daily run-off estimates can be made along the lines suggested, but such results should always be checked against determinations made by other methods and the most reasonable value adopted, unless reliable long-term flow records are available near-by.

The purpose of this discussion has been to offer additional suggestions which may prove helpful in what is at best a very difficult and uncertain problem. Changes in run-off régime, as in rainfall, are abrupt, and result from many factors operating some against, some with, each other. In the last analysis, only general tendencies or types of years can be predicted and, even after the most intensive study, departure from the expected will be encountered.

In conclusion, the writer feels that even with the relatively short-term run-off records available in the West, streams with long-term records can be found elsewhere, which will furnish a basis for extending the short-term record. Results obtained in this manner should furnish an excellent check on determinations made entirely from rainfall data.

C. F. Marvin, Esq.* (by letter).†—On behalf of the U. S. Weather Bureau, the writer may state that Mr. Grunsky's request for a change in the annual publication of meteorological data, in that the annual summaries should apply, throughout the whole country, to a climatic year and not to a calendar year, was received and considered in a sympathetic manner. Shortly after its receipt, letters requesting expressions of views on the subject-matter therein were sent to various persons and organizations. The response has not been as full and satisfactory as could be desired, and, therefore, a second inquiry and questionnaire is being distributed to recipients of the *Monthly Weather Review*, in the hope that discussion on this important proposition will be stimulated.

As Mr. Grunsky's request appears to have been based mostly on the needs of students of rainfall in California, it has been thought desirable to secure a wide expression of views, and, therefore, correspondence and discussion on the subject, on the part of engineers, are solicited by the Weather Bureau.

Rudolph Hering,‡ M. Am. Soc. C. E.—The question of rainfall and run-off studies has been before engineers for a great many years. About forty years ago, when the speaker had to face the problem for the first time seriously, it seemed almost hopeless to obtain what an engineer likes—exactness—some

* Chf., U. S. Weather Bureau, Washington, D. C.
† Received by the Secretary, October 4th, 1921.
‡ Montclair, N. J.

formula on which one could positively rely. About that time, the speaker was also confronted with the use of formulas for similar physical data as aids in the solution of problems, and it sometimes seemed hopeless to use formulas for any series of occurrences.

This remark was once made about some pile-driving experience to the editor of *Engineering News,* who requested something about that particular subject for publication, which information was subsequently furnished him. This was in regard to the Fairmount Bridge in Philadelphia, Pa. There was trouble about the piling, and the question arose as to whether or not the engineer could be sure that the pile was going to hold up the weight to be placed on it when driven with a certain weight of hammer and a certain distance of fall. The speaker began to collect all the formulas available in America, England, France, and Germany on the subject, with the thought of finding out what those various formulas really meant. A heavy and a light hammer, and a high and a low fall were assumed and the results were worked out. These results were published in *Engineering News* (1889), and the late Arthur M. Wellington, M. Am. Soc. C. E., one of the editors, deduced afterward from those formulas the one now known as the Wellington formula.

Gen. McAlpine deduced the formula known as McAlpine's formula from experiments made in the Brooklyn Navy Yard. He gave it to the world without any limitations, and it has been used by engineers. It will be seen from that article in *Engineering News,* assuming a possible case of pile-driving, that the more the pile was pounded the less it would carry; in other words, the result was negative.

Now, of what use is such a formula? In fact, many formulas are questionable. The speaker has suggested that every one offering a formula should do so as in calculus, when the two limits to the integral sign are given, meaning that this formula is applicable within those limits and is no good outside of them.

One reason why Kutter's formula interested the speaker was because it comprised all cases from a 1-in. pipe to the Mississippi River. All the gaugings used were between those two limits, and the formula works astonishingly well under all circumstances if the coefficient of roughness is judged correctly.

This shows that the use of a formula by a practical engineer is somewhat dangerous, unless he knows the data from which the formula has been derived. In that case, of course, engineers have no better guide than a formula, because it practically gives them a curve between known facts, and is safe to use; beyond that, it is not safe.

Another case was published about 1878.* There was a discussion on the same subject as that presented by Mr. Grunsky in his paper, and a formula for run-off was presented. It was shown in the discussion how easy it was to obtain absurd results; and the speaker presented another method of getting those results, namely, by plotting curves from actual gaugings under different conditions. It was felt that those limited curves which were established from known facts, would always be reliable, whereas a formula representing a curve of perhaps infinite length, might lead to great error. Data had been collected

* *Proceedings,* Engrs. Club of Philadelphia, Vol. 1 (1880), p. 146 *et seq.*

of the run-off for many streams in America, Europe, and India, where heavy rain storms and floods occur, giving the maximum flood discharges down to small areas, such as the municipal engineer finds in sewerage problems.

Engineers should feel grateful to Mr. Grunsky for having compiled so many data and made such a thorough study of a very important subject which in the collection of data has by no means as yet been exhausted. He has given sufficient information to permit an engineer to proceed with more confidence in the results than without it.

In the Philadelphia Water Department, from 1883 to 1886, some experiments were made with some reference to this subject. The streams were gauged with automatically registering gauges. Not only was the total rainfall measured, but there was also automatic continuous registration. After the speaker left that Department, 36 years ago, these observations were continued, and if the data were properly compiled, they would furnish an unusual amount of information of the run-offs from areas of different topography varying from 100 to 400 sq. miles, extending over a period of from 25 to 30 years. The areas differed in their physical character, some being hilly and some flat; some were much wooded, and others were open agricultural areas.

In order to be able to interpret intelligently these run-offs, with the permission of the Chief Engineer of the Department, the late William Ludlow, M. Am. Soc. C. E., a few acres of land—about half a dozen—with different physical characteristics were staked off. For instance, one was very steep and wooded, another steep but plowed agricultural land; then, again, there was a very flat area wooded and one that was open.

The rain was measured in a centrally placed gauge, and the run-off was measured by a water meter placed at the foot of the area, so that it made an interesting and valuable comparison. The results were never published, and the speaker does not know what became of them, but the original data must still be in the files of the Philadelphia Water Department.

Surveys were made of more than 400 sq. miles, and all the wooded and open areas, roads, and even every house in the area, were located, so that its physical character could be determined very closely. Topography over the whole area was taken with 10-ft. contour intervals. Therefore, data for a very interesting study of run-off may be found there recorded.

OLIN H. LANDRETH,* M. AM. SOC. C. E.—The membership of the Society, and the Profession at large, are under great obligations to Mr. Grunsky for his repeated valuable contributions to the technical literature on hydrology, including this interesting and valuable undertaking to derive empirical rules relating to rainfall and run-off on the Pacific Slope.

In referring† to the fact that the ratio of run-off to rainfall is greater on high mountainous areas than at low altitudes, the author attributes the difference, not to "the character of the surface of the water-shed", but rather to the smaller loss from evaporation in high mountainous areas, due to the lower temperature, and, consequently, the greater proportion of rainfall remaining as run-off.

* New York City.

† *Proceedings*, Am. Soc. C. E., September, 1921, pp. 222–223.

The fact is not questioned that in high altitudes a greater ratio of run-off to rainfall prevails than in low altitudes, nor is it doubted that the smaller loss from evaporation in the higher altitudes constitutes the main, if not the only, cause of this difference; but, perhaps, it may be questioned whether the lower temperature in mountainous altitudes is the only cause for the reduced loss from evaporation.

In general, evaporation per unit of superficial area from the free surface of water, and at much lower rates from the surface of snow and ice, mainly depends on, and varies with, three elements: (1) the temperature of the evaporating surface or film; (2) the humidity and the degree of agitation of the air above the evaporation surface; and (3) the length of time of exposure to evaporation. If, instead of a free water surface, the evaporation is from the surface of soil supplied with moisture from ground-water, an additional factor also affects the evaporation rate, namely, (4) the rate at which water is drawn up to the ground surface by soil transpiration, which, in turn, depends on the minimum transpiration capacity of the soil for a unit of depth, and also on the depth of the ground-water below the surface of the soil.

At high mountainous altitudes not only are temperatures generally lower than on the lower plain areas, but the average time of exposure to evaporation is also less, because of the average steeper slopes and resulting higher velocities, both for the free running water and for the ground-water moving down the slopes by percolation.

The average time of exposure to evaporation should vary approximately inversely as the square root of the slope. Thus, with other controlling elements the same, but with comparative slopes of 2° and 8°, respectively, the latter should give average times of exposure to evaporation about one-half those of the former, and as evaporation varies directly with the time of exposure, the evaporation should be about one-half as much on the steeper as on the flatter slope. Also, the generally greater coarseness of soil grains on mountainous areas, as compared with flat areas, favors a more rapid rate of percolation of ground-water and, therefore, still further tends to shorter times of exposure and consequent smaller evaporation loss.

In addition, even with equal temperatures, the evaporation loss from the ground surface should be less on mountainous than on lower plain areas, due to the lesser capillarity and, therefore, lesser transpiration capacity of the coarser mountain soils, as compared with the finer grained soils of the lowlands.

From the foregoing, it would appear that the generally lower temperatures in mountainous areas should not be assumed to be the only cause of the observed lower evaporation losses, but rather that the differences in average slopes and in average soil fineness should be recognized as playing parts—and, perhaps, important parts—in causing the smaller evaporation losses in mountainous as compared with low-level areas.

If this is true, then the desirability should evidently be considered of modifying Equations (13) and (14), which are offered by the author as means of predicting the run-off in mountainous regions from that at low altitudes. These equations at present contain only one factor to account for the difference

in run-off, namely, f, the mean temperature throughout the water-shed in question during the "wet season" from December to May.

If, however, it should be decided not to modify the present form of Equations (13) and (14), then, notwithstanding the factor, f, is still retained and used in its defined meaning, the empirical co-efficient, C, when evaluated from a sufficiently wide range of observations on the rainfall, P, and on r and r', the run-off from the low-level and mountainous areas, respectively, will inevitably contain the effect of all the elements which really cause the observed difference in the ratio of run-off to rainfall in the high and low areas, respectively, even though no terms are introduced into the equations to represent such other elements than temperature.

MEMOIRS OF DECEASED MEMBERS

Note.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

ELIOT CHANNING CLARKE, M. Am. Soc. C. E.*

Died May 4th, 1921.

Eliot Channing Clarke, the son of the late Rev. James Freeman and Anna (Huidekoper) Clarke, was born on May 6th, 1845, at Boston, Mass. The first ten years of his life were spent in Boston and in Meadville, Pa., the home of his mother's family. In 1855, his family settled in Jamaica Plain, Mass., where he attended the public schools and was fitted for college at the Eliot High School. He was graduated from Harvard College in 1867, serving as Chief Marshal of his class.

Mr. Clarke then took special studies at the Massachusetts Institute of Technology, and in February, 1868, began civil engineering work on a bridge, then building, over the Mississippi River at Quincy, Ill. Among other works with which he was connected later were the bridge over the Mississippi River at Hannibal, Mo., the Decatur and East St. Louis Railroad, the Phœnix Iron Works, the Detroit River Tunnel, the Chicago Water-Works tunnels, and the Chicago sewerage system.

In July, 1876, he was appointed Engineer in Charge of the survey for a main drainage system for Boston, Mass. The project having been adopted, he began construction in 1877, and the system was completed and put in operation early in 1884. Later in that year, Mr. Clarke became Chief Engineer of the Massachusetts Drainage Commission which was appointed to design methods of preventing the pollution of the water in the Charles, Mystic, and Blackstone Rivers Basins. In 1886, he was appointed a member of a commission to devise a plan for preventing floods in the valley of Stony Brook.

In December, 1886, and until 1904, he managed mill properties at Lowell, Mass., having served at different times as Treasurer of the Lowell Bleachery and the Boott Cotton Mills. In 1904, he retired from active business and, thereafter, until his death on May 4th, 1921, he attended only to his private affairs, with an office and residence in Boston and a summer place in Tamworth, N. H.

Mr. Clarke was fond of country life, of riding, and of fine horses of which he owned many. He had served as a Governor of the Riding Club of Boston.

He was a Fellow of the American Academy of Arts and Sciences and its Treasurer for eleven years. He was also a member of the Massachusetts Natural History Society, the Massachusetts Horticultural Society, the Colonial Society, and the Corporation of the Massachusetts Institute of Technology. He had also served as a Trustee of the Massachusetts School for the Feeble-Minded, Trustee and Vice-President of the Provident Institution for Savings, and as a Director of the State Street Trust, and other companies.

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* Memoir compiled by Arthur T. Safford, M. Am. Soc. C. E.

In 1885, Mr. Clarke was awarded the Norman Medal by the Society for his paper "Record of Tests of Cement Made for Boston Main Drainage Works, 1878-1884"*. He had also published an account of the Main Drainage Works of Boston, and had written many other technical papers and discussions. Of his little treatise on "Astronomy", 12 000 copies were sold.

In 1878, Mr. Clarke was married to Alice de Vermandois Schier, by whom he had five children, three of whom survive him, namely, Susan Lowell Clarke, James Freeman Clarke, and Mrs. Charles Eliot Ware, Jr.

Mr. Clarke was elected a Member of the American Society of Civil Engineers on September 4th, 1878, and served as Director of the Society in 1889. He had also served as a member of the Special Committee of the Society on a Uniform System for Tests of Cement.

FRANCIS COLLINGWOOD, M. Am. Soc. C. E.†

Died August 18th, 1911.

Francis Collingwood, the son of Francis and Elizabeth (Kline) Collingwood, was born in Elmira, N. Y., on January 10th, 1834. His father, a watchmaker and jeweler, was a native of Manton, Rutlandshire, England, and came to America about 1814. His mother was from Elmira, N. Y., her ancestors having come to the United States from Germany, and having been among the early settlers in this country. Her father served in the Revolutionary War.

After attending the Academy in Elmira, Mr. Collingwood entered Rensselaer Polytechnic Institute, at Troy, N. Y., and was graduated therefrom at the head of his class in 1855, with the degree of C. E. He then went to Wisconsin, where he was occupied in railroad engineering work, and, later, became City Engineer of Elmira, N. Y.

For about ten years ending in 1869, he was engaged in the sale of scientific instruments, but left that business to accept a position as Assistant Engineer on the East River Suspension Bridge, New York City, where he remained until July, 1883. His work in connection with this structure had to do more especially with the sinking of the caissons and the building of the towers, the anchorages, and the New York City approach, which was designed entirely by him. He was one of the first engineers to sink very large caissons, and those used in laying the foundations of the bridge towers were the largest ever constructed at that time. When this vast work was completed Mr. Collingwood was occupied in making extensive repairs to the Allegheny Suspension Bridge, at Pittsburgh, Pa., which had suffered from rusting of the cable wires at the anchorages.

Beginning in 1885, he became a regular contributor to the *Sanitary Engineer,* and, at the same time, he opened an office in New York City as Consulting and Expert Engineer. In this capacity Mr. Collingwood gave testimony in many cases of back-water, infringements of water rights, masonry failures, arches, etc., advised in cases of flood protection, was consulted in the

* *Transactions*, Am. Soc. C. E., Vol. XIV (1885), p. 141.

† Memoir compiled from information furnished by W. P. Mason, M. Am. Soc. C. E., and on file at the Headquarters of the Society.

case of the Johnstown Flood, was an Expert Examiner respecting frauds in the New Croton Aqueduct, and was employed as Consulting Engineer in important sewer work.

He wrote many papers concerning bridge work, particularly caisson, tower, and anchorage work, which have been published by the Society.* He also prepared a paper on the repairs to the Allegheny Suspension Bridge for the Institution of Civil Engineers for which he was awarded the Telford Medal and the Telford Premium.†

Mr. Collingwood died on August 18th, 1911. He was married on June 5th, 1860, to Eliza W. Bonnett, of New York City, the daughter of Daniel and Margaret Bonnett, who survived him.

He was a member of the American Institute of Mining Engineers, New York Academy of Science, Institution of Civil Engineers, American Geographical Society, Metropolitan Museum of Art, New York Microscopical Society, American Association for the Advancement of Science, and American Institute of Architects.

Mr. Collingwood was elected a Member of the American Society of Civil Engineers on March 3d, 1869. He served as a Director of the Society from 1873 to 1876, as Secretary from 1891 to 1894, and he instituted and endowed the Collingwood Prize for Juniors in 1894.

JOHN BAILLIE HENDERSON, M. Am. Soc. C. E.‡

Died February 15th, 1921.

John Baillie Henderson was born in London, England, in 1836. His father was a native of Ross-shire, and his mother of Ayrshire, Scotland. He was educated at Stanmore, Lanark, and Glasgow, and when a young man went to Australia where he began his professional career by work on railroads, roads, and bridges.

In April, 1878, Mr. Henderson went to Queensland and was appointed Resident Engineer on the completion of the Coliban System, then the largest water-works system in Victoria, the cost of which was more than £1 000 000 and from which the mining towns of Taradale, Castlemaine, Chewton, and Bendigo are supplied with water for domestic, manufacturing, and mining purposes. His duties in connection with this work included the revision of the System, which made new surveys and new designs necessary, and these were all executed under his direction. Previous to this work he had completed the Geelong System, the cost of which was £300 000.

In 1883, Mr. Henderson was appointed Government Hydraulic Engineer of Queensland, and he will always be remembered for his activities in connec-

* "A Few Facts About the Caissons of the East River Bridge", *Transactions*, Am. Soc. C. E., Vol. I (1872), p. 353; "Foundations for the Brooklyn Anchorage of the East River Bridge", *Transactions*, Am. Soc. C. E., Vol. III (1874), p. 142; Vol. IV (1878), p. 205; "Further Notes on the Caissons of the East River Bridge", *Transactions*, Am. Soc. C. E., Vol. II (1873), p. 119, etc.

† "On Repairing the Cables of the Allegheny Suspension Bridge at Pittsburgh, U. S. A.", *Minutes of Proceedings*, Inst. C. E., Vol. LXXVI (1883-84), p. 334.

‡ Memoir compiled from information supplied by E. J. T. Manchester, M. Am. Soc. C. E., and on file at the Headquarters of the Society.

tion with the development of the artesian bore system which made so great a difference in the value of the western lands. He was among the first engineers to give serious consideration to the power possibilities in these released underground waters, and he believed the supply was not inexhaustible. It was largely through his warnings of the danger of exhaustion that the Government assumed control of the sinking of bores. He was also interested in plans for irrigation and stream gauging which had been allotted to his Department.

Mr. Henderson retired from the position of Government Hydraulic Engineer of Queensland at the end of 1916, but, in spite of his eighty-five years, always took an active interest in public affairs.

He died at his home, Monkira, Hawthorne, Brisbane, Queensland, Australia, on February 15th, 1921, after an illness of a few days, and is survived by his wife, two sons, Hector and Ernest Henderson, and one daughter, Mrs. E. H. Pike.

Mr. Henderson was a member of the Institution of Civil Engineers, Institution of Mechanical Engineers, Society of Engineers, Association of Engineers and Shipbuilders of Scotland, Royal Geographical Society of Australasia, Royal Society of Victoria, Institution of Surveyors and Engineers, Victoria, and the Engineering Association of New South Wales.

Mr. Henderson was elected a Member of the American Society of Civil Engineers on June 4th, 1890.

WILLARD ATHERTON NICHOLS, M. Am. Soc. C. E.*

Died August 25th, 1921.

Willard Atherton Nichols, the son of Dr. George Henry and Sarah (Atherton) Nichols, was born at Standish, Me., on August 22d, 1844. His ancestors were distinguished Colonial and Revolutionary New Englanders. After preparatory work at the Boston Latin and English High Schools, he was graduated from Harvard University (Lawrence Scientific School) in 1865 as Civil Engineer, with the degree of B. S.

His first work was, for a short time, as Assistant Engineer of the Bureau of Sewers of the Croton Aqueduct Department of New York City. He left that position to build the Sullivan County and Erie Railroad in Pennsylvania. When that railroad was completed, Mr. Nichols went to Maine and, as Chief Engineer, took charge of the construction of the railroad then known as the European and North American Railroad, from Bangor, Me., into New Brunswick, which railroad has since been absorbed and consolidated in the Boston and Maine and other roads.

In June, 1876, he was offered and accepted the position of First Assistant Engineer in the Department of Docks of New York City. He fulfilled the duties of that position efficiently and with distinguished ability until his health failed, and, in 1890, the Department granted him a leave of absence in order that he might recover his health and resume his work.

* Memoir prepared by George S. Greene, Jr., and Charles H. Myers, Members, Am. Soc. C. E.

Mr. Nichols however did not return to this work, and having been advised or rather ordered to seek a milder climate, he went to Redlands, in Southern California, where he bought land and planted an orange grove which was successful. He also practiced his profession there, as his health and strength permitted, and gave much advice from his knowledge and experience to the people in the vicinity by whom he was very highly respected and very much liked.

Mr. Nichols was a strong and able man and a good citizen, and one of his noticeable qualities was the evenness of his temper under all circumstances.

He did not marry, and is survived by a twin sister, Mrs. Elizabeth K. Hills, of Marblehead, Mass., and two nephews, Henry Atherton Nichols, a banker of Cambridge, Mass., and John Gilman Nichols, a New England manufacturer.

Mr. Nichols was a member of the Society of Colonial Wars, of the Sons of the Revolution, and of the Massachusetts Society of Mayflower Descendants. He was a member of the Unitarian Church of Redlands, Cal., a Trustee of the Smiley Public Library, and a member of other local clubs.

Mr. Nichols was elected a Member of the American Society of Civil Engineers on May 7th, 1873.

WILLIAM JAMES DAVIS, Assoc. M. Am. Soc. C. E.*

Died September 2d, 1921.

William James Davis was born in Lachine, Que., Canada, on June 26th, 1884. He was the son of Thomas Davis, of Lachine, and Mary Sproul, of Maxville, Ont., Canada. He attended the public schools in Montreal West and the Montreal High School, and his technical education was obtained in a special class conducted in Montreal by W. Chase Thomson, M. Am. Soc. C. E., who is now a Consulting Engineer in that city.

In May, 1905, Mr. Davis began work as a Templet Maker for the Locomotive and Machine Company of Montreal, which Company had at that time just begun the fabrication of structural steelwork as an adjunct to its tank work. To him, a job was not something to be "held down", but an opportunity for work, and this optimistic energy, which was one of his characteristics, showed itself in the templet shop from the very first.

During the fall of 1907, he left the Locomotive Company to enter the Drawing Office of the Dominion Bridge Company's Head Office at Lachine. Before long, he proved his worth in this field also and was promoted to be a Checker. In this capacity, Mr. Davis had considerable experience in structural steelwork for railway and highway bridges, swing bridges, office buildings, mill buildings, conveyor trestles, wharf sheds, elevators, theatres, churches, and miscellaneous structures.

In 1913, the Dominion Bridge Company decided to open a Drawing Office at its new branch in Winnipeg, Man., and Mr. Davis was chosen to take charge of this new Drafting Department. Within two months he was made Construction Engineer of the Winnipeg Plant and acted as Assistant to the Western

* Memoir prepared by F. P. Shearwood, M. Am. Soc. C. E.

Manager, George E. Bell, Assoc. M. Am. Soc. C. E., whose headquarters were at Winnipeg. It was the formative period in the Company's business in that place and was a time of keen competition. Probably no one except those on the ground realized the tireless energy with which Mr. Davis threw himself into every detail of the Winnipeg Plant. The results proved the good judgment which had been shown in his appointment.

Directly following the entry of Great Britain into the World War, a period of reorganization took place, and, in 1915, Mr. Davis returned to the Head Office of the Dominion Bridge Company, at Lachine, as Assistant Works Manager, a position for which he was well suited by his experience in the shop, in the office, and on the Executive Staff at Winnipeg. In his new capacity he showed his ability to make things move, and his example was an incentive to those with whom he came in contact in the works.

During the summer of 1919, Mr. Davis left the Dominion Bridge Company to accept a position as Engineer with the Wayagamack Pulp and Paper Company, at Three Rivers, Que. In this capacity, he acted as Chief Draftsman, Plant Engineer, and Assistant to the President. Owing to a decrease in the paper business within the last few years, the Engineering Staff was reduced appreciably, but the Company knew Mr. Davis' worth and retained his services. He designed and carried out considerable extensions to the machine-room and other buildings for the Company, his work including the approval of contractors' drawings, the design of foundations, and the superintendence of the construction of new buildings and of repairs to existing conveyors and bridges.

In the latter part of August, 1921, he took a well earned holiday, with some companions, in the timber limits of the Wayagamack Company, on one of the tributaries of the St. Maurice River. On September 2d, he went out with a guide on a small body of water called Lake Cutaway, and, through a slight accident, the canoe was capsized. It is thought that although he was not a good swimmer, he could have saved his own life had he not been anxious to save that of the guide also. The guide survived, but Mr. Davis sank in 40 ft. of water. His body was recovered after two days, sent to Three Rivers, and, from there, to Montreal where the funeral took place. Besides his widow and two daughters, Mr. Davis is survived by his father and mother, three brothers, and a sister.

On September 6th, 1911, he was married to Miss Florence Nutter, of Montreal. The marriage proved a singularly happy one, and their home was known to many friends on account of the marked hospitality of the host and hostess.

To all his business acquaintances and friends, Mr. Davis was known as "Bill". He was a man among men, a good sport in the very best sense of the word, and he could and did look every one straight in the eye. He was active in almost every sport, but particularly so in curling and bowling. Already a challenge shield has been presented to the Three Rivers Bowling Club by the Bowling Club at Montreal West, to be known as the "Bill Davis Shield". It is only one evidence in many of his work for good clean sport. His life—thirty-seven years—seems short, but it cannot be called incomplete. His con-

sistent and intense record is an example not surpassed by long life and is a challenge to all.

As early as 1912, he became connected with the Canadian Society of Civil Engineers, now the Engineering Institute of Canada.

Mr. Davis was elected an Associate Member of the American Society of Civil Engineers on August 31st, 1915.

THOMAS GEORGE ELBURY, Assoc. M. Am. Soc. C. E.*

DIED JULY 6TH, 1921.

Thomas George Elbury, the son of Edward and Jane (Simmons) Elbury, was born at Bristol, England, on February 24th, 1862. His family for many generations had been engaged in the manufacture of pottery and clay products. During his boyhood, besides going to school at St. Lukes, Bedminster, Bristol, he was being educated to carry on the established business of his ancestors.

He showed little inclination for this work, however, and resolved to fit himself for a profession. At the age of fifteen he started out to earn his own living, and by working half the day, and going to school the other half, he laid the foundation for his future career.

After considering the opportunities offered young men in his native country, Mr. Elbury decided that a greater future awaited him in the United States; in accordance with this decision, he left England for the United States in August, 1883, and, two weeks after his arrival, he settled in Cleveland, Ohio. Working by day and going to school at night, he continued his studies and, in spite of many obstacles, he qualified for his chosen profession, Civil Engineering.

In 1885, Mr. Elbury moved to Kansas and was appointed Assistant Civil Engineer of Barber County. A few years later he was made Deputy Surveyor of Kingman County. In 1895, he was elected County Surveyor of Reno County, which position he held until 1905.

From 1902 to 1909, Mr. Elbury served as City Engineer of Hutchinson, Kans., where his work included the design of the Hutchinson Drainage Canal. In addition to other duties, he contracted and made preliminary surveys for the Northern Oklahoma Railway in 1902, and, in 1904, he was employed as Chief Engineer of the Gulf, Hutchinson, and Northwestern Railroad.

His last engineering work was the construction of the dam for the Columbia and Okanogan Rivers at Kennewick, Wash., and the dam at St. Maries, Idaho, after which he retired from active engineering work.

For two years before his death, Mr. Elbury was associated with his son, in the publication of a technical paper, the *Big-4-Railroad Record,* in San Francisco, Cal.

In April, 1920, he visited his old home in England. Shortly after his return to San Francisco, he became seriously ill with a malady that developed into pneumonia, of which he died on July 6th, 1921.

* Memoir compiled from information furnished by Edward John Elbury, Esq., and on file at the Headquarters of the Society.

On August 21st, 1889, Mr. Elbury was married to Alice A. McKinnis, of Nashville, Tenn., who, with a son, Edward John Elbury, survives him.

He was a man of excellent character and was held in high esteem by his many friends and associates.

Mr. Elbury was elected an Associate Member of the American Society of Civil Engineers on May 6th, 1908.

EDGAR MILLER GRAHAM, Assoc. M. Am. Soc. C. E.*

DIED MAY 14TH, 1921.

Edgar Miller Graham was born in Buffalo, N. Y., on June 26th, 1881. He was the son of M. W. Graham, of Franklin, Pa., and Helen R. (Hall) Graham, of Ashtabula, Ohio. He obtained his preliminary education in the public schools, High School, and at Underwood Institute, at Buffalo, with the intention of entering the Massachusetts Institute of Technology. Instead of entering a technical school, however, he immediately started to work. He acquired his technical education by studying standard textbooks, with the assistance of technically educated friends. He also completed the Railroad, Municipal, Hydraulic, and Electrical Engineering Courses of the International Correspondence Schools.

In 1900, Mr. Graham began his engineering career as Chainman for an engineering firm in Buffalo, N. Y. His rise was rapid, for during the same year he served successfully as Transitman, Draftsman, and Chief of Party on work for the firm. Most of this work was in connection with the staking out of the large buildings for the Pan-American Exposition and also field engineering in connection with the Dakota and Great Eastern Grain Elevators, which were among the largest in the United States, the latter being one of the first reinforced concrete elevators in Buffalo.

During part of 1901, Mr. Graham was engaged in the Engineering Department on the Eastern Division of the New York Central and Hudson River Railroad. Later in 1901, and until 1903, he was in the service of the Lackawanna Steel Company on the construction of its large plant at Buffalo. During this time, he served as an Assistant in the Estimate Division, as Chief of Party on laying out all kinds of work, and as Assistant Engineer to the Chief Engineer.

Later in 1903, he was in charge of the design and construction of several miles of paving and sewers at Clearfield, Pa., and, in 1904, he was employed by the Harbison, Walker Refractories Company, at Pittsburgh, Pa., in the Sales, Operating, and Mining Departments.

From 1905 to 1908, Mr. Graham was in the service of the Buffalo and Susquehanna Railway Company, first, as Draftsman, then Chief Draftsman, and Assistant Engineer in charge of the office of the Assistant Chief Engineer at Buffalo. In 1907, he was made Assistant Chief Engineer, in charge of the Buffalo Office and of all engineering and construction work on 90 miles of railway from Wellsville, to Buffalo, N. Y.

* Memoir prepared by Milton Leon, Assoc. M. Am. Soc. C. E.

From 1908 to 1910, Mr. Graham was engaged in private practice at Buffalo. In the latter year, he moved to Oklahoma, and from that time until his untimely death, he maintained an office in Muskogee, Okla., as a Consulting Engineer. During this period of eleven years, he was very active in engineering and construction work in Oklahoma and the adjoining States. He made a number of investigations and reports for proposed railroads, acted as Consulting Engineer for several towns, and made several valuations of railroad and public service properties.

As Chief Engineer, he had charge of all engineering work of the Muskogee Electric Traction Company for 11 years. In the same capacity, he located the Webbers Falls Railroad and built the first 10 miles, all that was ever built. Mr. Graham was Consulting Engineer and Director of the Cushing Traction Company. In the capacity of Chief Engineer of the Cushing Construction Company, he had full charge of the location and construction of the lines of steam railroad built in the Cushing, Okla., Oil Fields. For the past couple of years, he had been making a special study of highway construction and paving in connection with the "Willite" process of paving.

Mr. Graham met his death in an automobile accident near Muskogee, Okla., on May 14th, 1921. In turning aside to avoid a wagon, his machine plunged into a deep ditch, and he was killed instantly.

In 1907, he was married to Gertrude Thornton, the daughter of Mr. and Mrs. James Thornton, of Wellsville, N. Y., who, with his father, survives him.

Mr. Graham was a Thirty-second Degree Mason, a member of the Shrine, Benevolent Protective Order of Elks, Jovian League, Little Rock Engineers Club, and the American Association of Engineers. As President of the Local Chapter of the American Association of Engineers and as Vice-President of the Oklahoma Section of the Society, Mr. Graham was very active in all work for the betterment of engineering and advancement of the professional engineer.

He was held in highest esteem by his friends and associates, and was one of the best known engineers in the State.

Mr. Graham was elected an Associate Member of the American Society of Civil Engineers on October 5th, 1909.

CHARLES RAYMOND LARKIN, Assoc. M. Am. Soc. C. E.*

Died August 30th, 1921.

Charles Raymond Larkin was born in Philadelphia, Pa., on November 6th, 1891. He was educated in the public schools of that city and was graduated from the North East Manual Training School in 1910. He then entered Villa Nova College, from which he received the degree of B. S. in Civil Engineering in 1914, and the degree of Civil Engineer in 1919.

Mr. Larkin began his engineering work with the Union Paving Company, as Superintendent and Highway Engineer, and many sheet asphalt streets, in the City of Philadelphia and its vicinity, were laid under his direction.

* Memoir prepared by H. B. Floyd, Esq., Philadelphia, Pa.

In August, 1916, Mr. Larkin was appointed Assistant Engineer in the Bureau of Health, City of Philadelphia, under J. A. Vogelson, M. Am. Soc. C. E., Chief of the Bureau, and, in this capacity, all construction and alterations undertaken in the hospitals and other institutions under the Bureau were done under his supervision.

In 1918, he was given a leave of absence from the Bureau of Health and enlisted in the Army, attending the Training School at Camp Joseph Johnston, Jacksonville, Fla. He was commissioned as a Second Lieutenant in the Quartermaster Corps on December 6th, 1918, but was retired to the Officers' Reserve Corps and again resumed his duties with the Bureau of Health where he remained until his death on August 30th, 1921.

Mr. Larkin was married on November 17th, 1920, to Katharine E. Lochrey, at Jamaica, Long Island, N. Y., and is survived by his widow, his father, and a brother.

He was a man of sterling character and exceptional ability and showed an earnestness and zeal in everything which he undertook that inspired the confidence of his associates and gave promise of a brilliant career. He will always be remembered as one of those rare personalities whose bigness of heart and breadth of spirit endeared him to all.

Mr. Larkin was elected a Junior of the American Society of Civil Engineers on January 14th, 1918, and an Associate Member on June 1st, 1920. He was also a member of the Engineers Club of Philadelphia and the Henry H. Houston Post No. 3 of the American Legion.

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AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

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Vol. XLVII. DECEMBER, 1921. No. 10.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

WINTER OVERFLOW FROM ICE GORGING ON SHALLOW STREAMS

By J. C. Stevens,* M. Am. Soc. C. E.

Synopsis

Little has been written on the subject of ice gorging since the report of the St. Lawrence River Commission and the construction of the ice barrier to protect Montreal, Que., Canada. The overflow of certain Western streams, caused by ice gorging during the winter months, presents quite different phases of this question.

The occurrence discussed in this paper is not the overflow caused by ice jams brought about, during the spring break-up, chiefly by blocks of crystalline surface ice being arrested at some constricted portion of the river channel and thus forming a temporary dam.

The ice gorge herein discussed occurs during the coldest part of the winter on streams too turbulent to permit the formation of crystalline surface ice; however, frazil and anchor ice are formed in such quantities that the stream becomes a viscous mixture. This results in a rise of the water surface and if the banks of the stream are too low to accommodate this increase of stage, overflow is inevitable.

Increasing the flow of such streams during the winter months by the release of artificially stored water presents an entirely new phase of the problem and merits careful analysis, especially where the overflow may become a menace to life and property.

The facts gathered by the writer during an investigation of the winter overflow from Madison River, Montana, and the effect of storage reservoirs

Note.—Written discussion will be published in a subsequent number of *Proceedings*, and, when finally closed, the papers, with discussion in full, will be published in *Transactions*.

* Portland, Ore.

thereon, are also true of other streams with similar characteristics and uses and should be of more than passing interest in the field of hydraulic engineering.

Statement of the Problem

Madison River rises in Yellowstone National Park and flows in a general northerly direction, uniting with Jefferson and Gallatin Rivers near Three Forks, Mont., to form the Missouri. It flows through two agricultural valleys locally known as the Upper and Lower Madison Valleys. In these valleys the river banks are low, and near the lower end of each valley the river divides and subdivides into a network of many brush-lined channels.

In these many channeled parts of each valley, during the cold winter months, ice gorges of varying characteristics are formed. These gorges frequently cause the river to leave its channel entirely and flow across the valley floor, occasionally driving the residents from their homes and leaving the valley covered with solidified frazil ice many feet in thickness.

In 1913, the Montana Reservoir and Irrigation Company completed the Hebgen Reservoir. In the summer this reservoir is operated for the purpose of supplementing the low-water flow of the Madison and Missouri Rivers for the benefit of the Prickly Pear Irrigation Project near Helena, and during the fall and winter for eight hydro-electric plants of the Montana Power Company on those rivers.

The winter of 1916-17 was one of exceptionally sustained, moderately low temperatures, during which an unusual quantity of frazil and anchor ice was formed. This resulted in ice gorges and extensive overflow of the agricultural lands in both valleys. The question arose as to whether the ice gorging and the overflow were augmented by the operation of the Hebgen Reservoir and of the Madison Reservoir and the power plants in the canyon between the upper and lower valleys (Fig. 1). Accordingly, there was instituted a thorough study of the ice gorging and winter overflow phenomena of Madison River both from a physical and an historical standpoint, as well as the effect of the operation of the reservoirs and power plants thereon. This was done in order to ascertain whether such operation was responsible wholly or in part for the overflow and to outline, if possible, some remedial measures.

Phenomena of Ice Formation

The temperature of a mixture of ice and water is always at 32° Fahr., as far as practical temperature measurements are concerned. It is a fact that small differences of temperature do exist; the range, however, is a matter of a few thousandths of a degree above or below the freezing point.

This condition is due to the latent heat of fusion of water. Whenever 1 lb. of water passes from the liquid to the solid state about 144 B. t. u. are liberated. Conversely, when 1 lb. of water in the form of ice passes to the liquid state, the same quantity of heat is absorbed in causing the molecular change from a solid to a liquid state and does not appear as a temperature

change in the body. Therefore, a mixture of ice and water maintains a balance of temperature; if it is subjected to a cooling influence, more ice is formed, which liberates just sufficient heat to keep the mixture at a constant temperature, and *vice versa*. It is of interest to note that from every acre of surface ice formed, 1 ft. in thickness, the same quantity of heat is liberated as in the burning of 18 tons of ordinary coal.

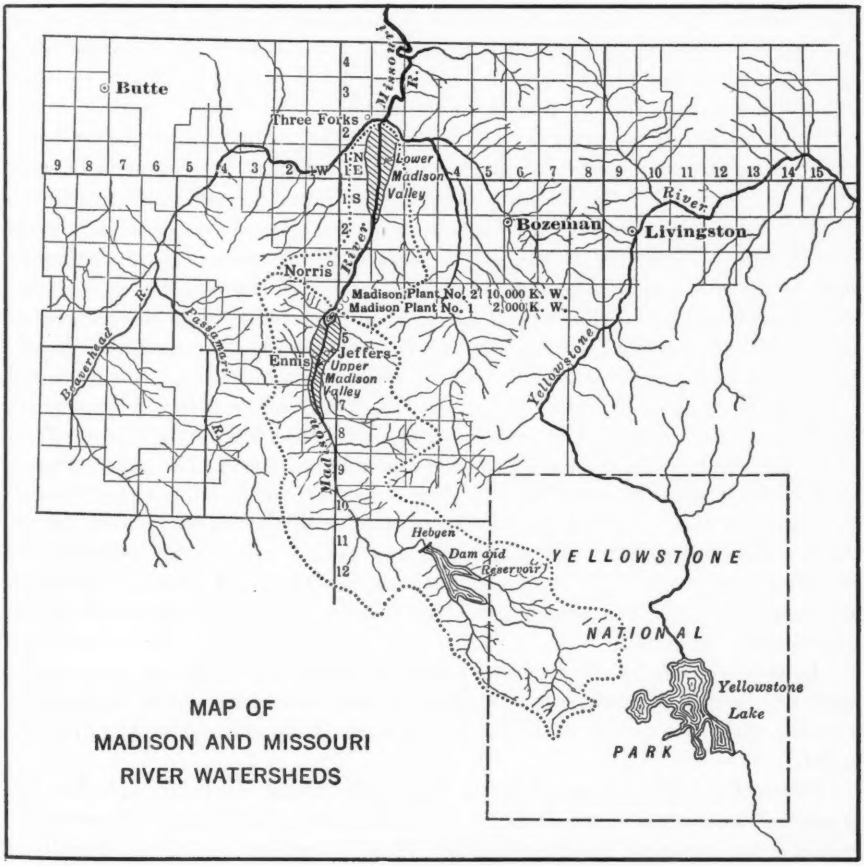

FIG. 1.

There is always a transfer of heat from a warm to a colder body. This transfer takes place by three processes, namely, radiation, convection, and conduction, all of which may be operating at the same time. The water in a river is cooled largely by convection. As the temperature of the air falls below that of the water, a transfer of heat immediately takes place from the warmer water to the colder air. The water at the surface is cooled by contact with the air and becomes denser, therefore, this layer of water sinks and is replaced by warmer water, thus setting up convection currents which gradually cool the whole mass.

When the body of water reaches a temperature of 39° Fahr., a reverse process sets in. At this temperature, water has its maximum density and expands as it is further cooled. Therefore, between 39° and 32° Fahr., the colder water remains at the surface in contact with the cold atmosphere.

Surface Ice.—As soon as the surface layer of water has reached a temperature of 32° Fahr., ice begins to form. From the water's edge the ice crystals begin to spread out over the surface, gradually extending toward the center of the stream. A nucleus of some kind is necessary for the formation of an ice crystal. Such nuclei may be tiny globules of dissolved air from the spray of tiny waves that break on the shore, snowflakes, particles of silt or sand, or any microscopic foreign matter in the water.

After a surface covering of ice is formed, the cooling of the water below it by convection virtually ceases, because the temperature gradient between the under surface of the ice and the water is indeed very slight. Continued cooling by conduction through the ice, however, does go on, but at a much slower rate. Thus, the thickness of surface ice increases at a continually decreasing rate because the quantity of heat conducted through the ice decreases as the ice increases in thickness. An accumulation of snow on the ice greatly diminishes the thermal conductivity of the surface covering and also greatly decreases the rate of ice growth formed from the under side. A substantial covering of ice usually prevents the growth of any other form of ice beneath it, that is, frazil and anchor ice cannot form, because further heat losses from the body of water and the river bed are practically stopped. The water beneath surface ice is a few thousandths of a degree above freezing temperature, except, perhaps, the film of water directly in contact with the under side of the ice sheet. In cold climates all modern power plants utilize this principle by securing a substantial reservoir above the power plant, and where so arranged, no matter how cold the weather, troubles from frazil ice in turbine wheels are practically eliminated.

In this respect the effect of a covering of surface ice on a lake or reservoir may be likened somewhat to the effect of the glass cover of a hothouse in keeping the temperature of the water beneath it slightly above the freezing point.

Frazil Ice.—This form of ice is frequently called slush ice, spicular ice, mush ice, and needle ice. It is also a surface ice, and appears whenever the water is too turbulent for the formation of solid surface ice. Great quantities of this ice may be formed during protracted cold weather. It floats and has the general appearance of snow in the water, but on examination is found to consist of small spicules or needle-like crystals which have a tendency to cohere in masses. The entire water area of the river may become impregnated with frazil ice, forming a mixture much more viscous than water, which causes a retardation of velocity with a consequent increase of stage. Favorable conditions for its formation are turbulent water with cold, strong, up-stream winds and cloudy weather.

Anchor Ice.—Except at its place of formation, which is on the bed of clear shallow streams, anchor ice can hardly be distinguished from frazil ice. The cause of its formation is the rapid radiation of earth heat from the river bed

into space. It forms most rapidly on dark-colored stones, during clear cold nights, and it will not form under bridges, in the shade of overhanging trees, during cloudy weather, and seldom or never under surface ice or in water which is heavily loaded with frazil ice or silt. It is purely a product of radiation and, in this respect, forms under the same conditions as hoar frost.

With a slight rise of temperature during clear, sunny days, this anchor ice will rise to the surface. In rising, it often lifts and carries away the rocks and gravel to which it originally adhered. In appearance, anchor ice is granular and somewhat resembles honeycomb. Once formed, it may grow rapidly by an accumulation of frazil ice adhering to it from the flowing water, and when floating in the river, it cannot be distinguished from a mass of frazil ice.

Causes of Winter Overflow

It has been stated by local authority that the Madison is the only river in Montana which overflows from ice gorges during cold weather. This statement is not true. The Ruby, Boulder, and many small streams behave exactly in the same manner as the Madison in this respect. Floods on the Yellowstone, Missouri, and other streams are frequently caused by ice jams during the spring break-up. Ice gorges from frazil and anchor ice also form during the winter months, but the overflow caused by the latter is limited to the lower lands adjacent to the river.

The Madison is probably the largest river in the State in which winter overflow conditions are so pronounced. The reasons are not hard to find. Madison River has a fairly steep gradient throughout its course. In the two valleys under consideration, the banks are low, the river is shallow and wide, and the bed is strewn with boulders, cobblestones, and gravel. There are many low, brush-covered islands and bars, which form in some places a veritable network of channels, which condition is the natural result of the flow characteristics of this river, combined with the slope of and the gravelly soil composing the valleys. The flow of Madison River is comparatively uniform, and as shown by Table 1 has only about one-fourth the range of the other streams.

TABLE 1.

Stream.	Locality.	Maximum flow, in second-feet.	Minimum flow, in second-feet.	Percentage of maximum flow.
Yellowstone	Huntley	47 900	1 060	2.2
Jefferson	Silverstar	16 500	320	1.9
Gallatin	Logan	5 400	178	3.3
Madison	Plant No. 2	9 500	1 200	12.6

Other things being equal, a stream with extremes of flow will have a deeper channel in its self-made valleys than one with a low range of flow. Therefore, the Madison, with its low range of flow, has banks averaging about 3 ft. in height through the valleys.

The water surface of a stream carrying large quantities of frazil and anchor ice is raised by reason of the reduction in velocity caused by the increased

viscosity of the flowing mixture. A condition is soon reached in which the river changes from a turbulent to a placid stream. If the banks are high enough this placid condition is reached before overflow occurs and surface ice is formed, which effectually prevents the further formation of frazil and anchor ice beneath it. Unfortunately, the banks of Madison River are so low that overflow often occurs before this condition is reached and is the principal reason for the extensive winter overflow from ice gorges.

The immediate cause of overflow is the formation of ice gorges induced by low temperatures, and such gorges invariably occur during a protracted period of cold weather. When the river water is cooled to the freezing point, frazil ice immediately begins to run. If the nights are cold and clear, anchor ice will form, and if the days are clear, these masses rise and float away. If the days and nights are cloudy, anchor ice does not form, but a greater quantity of frazil ice is created. During storms, snowflakes and frozen spray falling into the water form nuclei that aid in the growth of frazil ice. Thus, whether the atmosphere is clear, cloudy, or stormy, if the air temperatures are lower than freezing, the river, once the water has been cooled to the freezing point, is continually manufacturing ice, the rate depending on the air temperatures below 32° Fahr.

In the Upper Madison Valley, the ice gorges usually begin just above the upper end of the Madison Reservoir. This part of the valley is broken into a network of channels, and there are a great many low brush-covered islands and bars and innumerable obstructions to flow. In the Lower Madison Valley similar conditions exist.

With the accumulation of frazil and floating anchor ice these channels become completely choked and the river is virtually dammed with ice. Overflow is inevitable. First, the low sloughs and overflow channels are filled with water on which may form surface ice, and over which the river flows with its load of frazil. These channels soon are filled as high as the surrounding land, which causes the river to seek new outlets. In this manner the overflow spreads, and the water frequently leaves its natural bed altogether and flows over the surrounding bottom-lands. A water channel tends to choke itself with frazil ice; thus, the water is continually changing its path of travel and virtually raises the entire bottom to an irregular level of ice. While the cold continues, this frazil, exposed to the air, solidifies in huge masses, forming effective banks. There is practically no limit to the extent of overflow or ice accumulation that may occur as long as the critical degree of cold continues.

The surface ice at the edges is broken off by the rising water and floats down to add its volume to the gorge. These cakes frequently lodge edgewise and form an effective barrier for the accumulation of frazil and anchor ice. With a few days of rising temperature, the river begins to cut through and flows under the ice in its original channel, gradually clearing it of the accumulated ice. Oftentimes banks of frazil 7 or 8 ft. high are left along the river's edges.

Two types of ice gorges were recognized on Madison River, namely, that designated as the "bridging gorge", in which little or no overflow occurs, and that called the "overflow gorge".

FIG. 2.—TYPICAL GORGE OF THE "BRIDGING" TYPE.

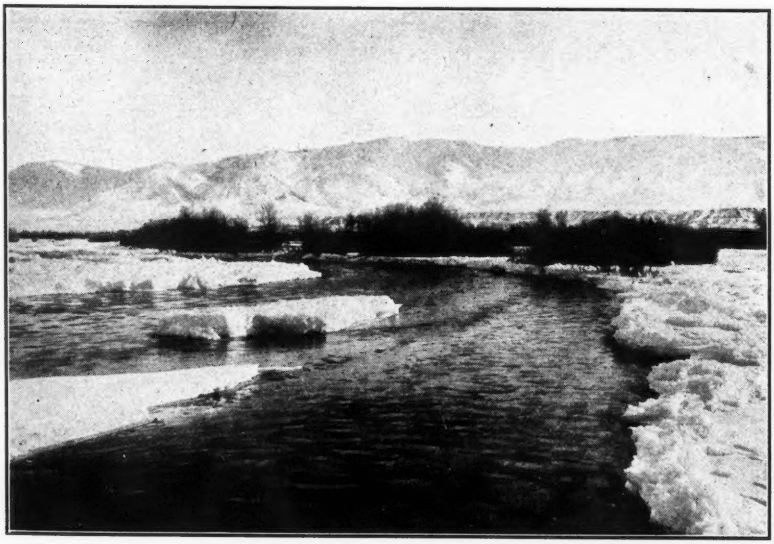

FIG. 3.—APPEARANCE OF RIVER AFTER ICE GORGE HAS BROKEN AND THE WATER HAS RETURNED TO ITS CHANNEL.

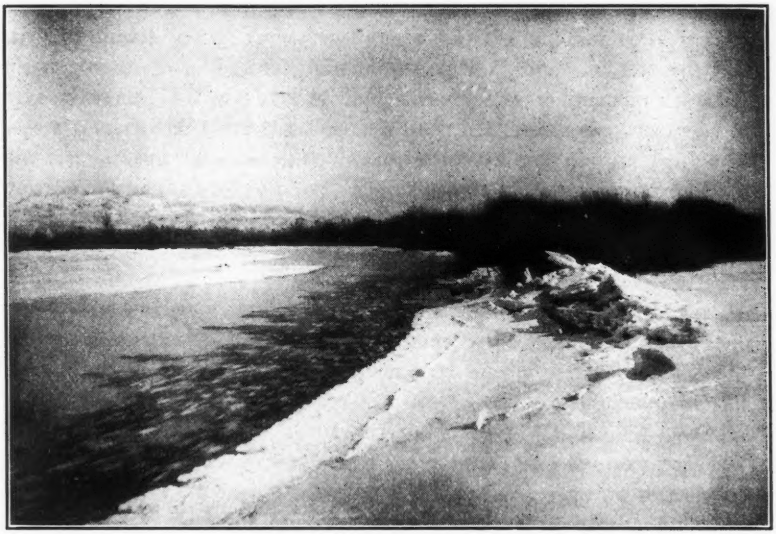

FIG. 4.—FRAZIL ICE FLOATING IN RIVER CHANNEL.

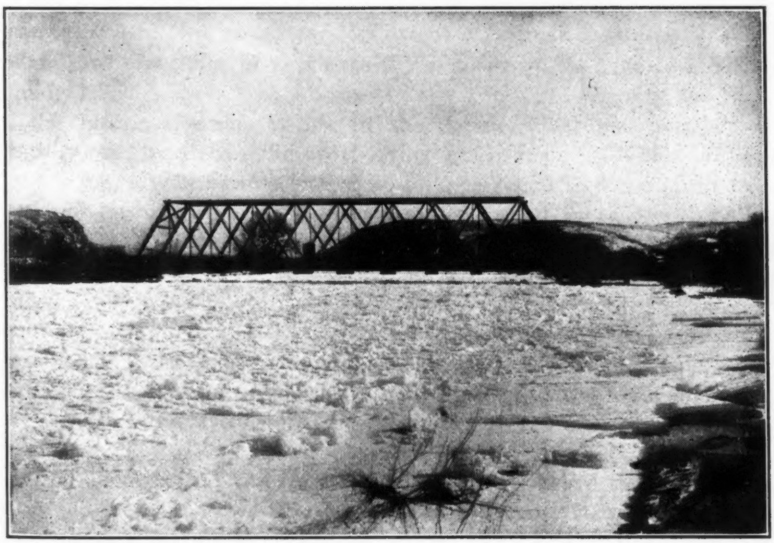

FIG. 5.—ICE GORGE OF THE "BRIDGING" TYPE ON GALLATIN RIVER.

The "bridging gorge" is caused by sudden and sustained extreme low temperatures, say, from –15° to –30° Fahr. This condition causes the maximum quantity of frazil ice to form, and the river is suddenly converted from a turbulent stream to a sluggish flowing mixture throughout its entire length. The river rises and is frozen over for long stretches of its length, thus preventing in these stretches the further formation of frazil and anchor ice. As long as the low temperature continues, the water flows beneath the ice bridge without overflowing its banks appreciably. Fig. 2 shows such an ice bridge with the water flowing beneath it.

The "overflow gorge" is caused by sustained moderate temperatures, say, between 15° and 25° Fahr. Such a condition leaves the river open practically throughout its entire length, causes considerable frazil and anchor ice, but not enough to form a "bridging gorge". Hence, the formation of ice continues unabated, and the many channeled lower part of the valley becomes the gathering ground for all the frazil, surface, and anchor ice formed in the river above.

Between these two distinct types of ice gorges there is every possible intermediate type, the one merging into the other, causing varying degrees of overflow. The river is thus known to overflow one place one time, and some other place another time, in fact, every possibility of gorging and overflow between the two extremes is known to have occurred. So capricious are these overflows that, during the winter, stock is never allowed to pasture in the bottom-lands. In the history of these valleys, thousands of head of cattle have been trapped and lost in the winter overflow.

By interviewing the old settlers of the valleys, it was found that ice gorges and overflows were to a greater or less extent of annual occurrence. Winters that stand out especially in this respect are those of 1867, 1875, 1883, 1898, 1910, and 1917.

In the Lower Madison Valley in the decade, 1890-1900, iceboats were successfully used by a colony of sport-loving Englishmen. The ice overtopped the fence posts, giving clear stretches 10 miles or more in extent. In fact, it appears that the limits of the 1916-17 overflow had at times been equalled if not exceeded.

Effect of Reservoir Operation

Apart from an increase in winter flow, the effect of a reservoir in such a stream is first to diminish the quantity of ice below it. For example, consider a turbulent river, 100 miles in length, with a valley at the lower end. In its natural condition the river pours into this valley all the ice formed throughout its length.

Now if a substantial reservoir is built 25 miles above the valley, it will constitute an ice barrier for the 75 miles of river above it, since no frazil ice can pass the reservoir. Moreover, the water from this reservoir is slightly above freezing temperature and frazil ice cannot form until the water has been cooled to freezing in the channel below. It is obvious, therefore, that after the construction of the reservoir the quantity of ice reaching the valley

will be limited to that formed in the lower 15 or 20 miles of river, whereas, prior thereto, 100 miles of river contributed its ice to the valley.

The first effect of increasing the flow is to diminish the quantity of anchor ice that may form, due to the increased depth of water reducing the quantity of heat that may be radiated directly from the bed of the stream.

The second effect is to diminish the turbulency and encourage the tendency of the water at freezing temperature, to remain on the surface. Thus, the formation of frazil ice is more likely to be confined to the surface of the moving water, which is not the case when the water is brought to a uniform freezing temperature by thorough mixing.

The third effect is to increase the quantity of frazil ice about in proportion to the increase in the area of the water surface.

The fourth effect is to increase greatly the transporting power of the current, thus enabling the river better to carry its icy load without gorging and, therefore, delay the formation of the gorge until a rise of temperature destroys the tendency of the ice to gorge. The increased flow tends to cut out the gorge quicker with a slight rise in temperature; in other words, the river becomes more sensitive to the temperature changes and the duration of the overflow is less than with the normal flow. For example, consider a typical cross-section of Madison River, as given in Table 2.

TABLE 2.

Gauge, in feet.	Mean depth, in feet.	Width, in feet.	Discharge, in second-feet.	Area, in square feet.	Mean velocity, in feet per second.	Transporting power.
2.0	1.84	177	880	325	2.60	1
2.5	2.19	186	1 470	425	3.45	6
3.0	2.80	195	2 350	545	4.31	21
INCREASE IN PERCENTAGE:						
2.0 to 2.5	19	5	67	31	33	500
2.0 to 3.0	52	10	167	68	66	2 000

It is stated in textbooks on hydraulics that the weight of bodies which can be moved by a current varies as the sixth power of the velocity, hence the increase in transporting power, corresponding to an increase of 67% in discharge, is 500%, and an increase of 167% in the discharge increased the transporting power 2 000 per cent. Of course, when the river becomes loaded with frazil ice, the velocity is somewhat reduced and the river rises, the flow remaining constant. With the increased flow and a loaded river, the full effect of the increase in transporting power is not felt, but, nevertheless, there is a decided increase in its transporting power and its ability to prevent and to cut out gorges. If the quantity formed varies as the surface exposed, there is only a 10% increase of frazil ice with 167% increase of discharge.

The change in the winter régime of a river by reason of the operation of reservoirs, results from five causes. The effect of four of these causes is to diminish the quantity of ice formed, and only one has the effect of increasing it; hence, the net effect of the construction and operation of reservoirs on the

Madison River was to diminish the quantity of ice and ice gorging. As applied to Madison River, this fact was abundantly verified by the testimony of old settlers still living in the valleys.

The next point of inquiry is whether a reduction in the quantity of ice might not increase in some manner the overflow, by making the general type of gorge conform more nearly to that of the "overflow gorge" in distinction to the "bridging gorge", as previously defined. This question is intimately associated with temperatures and windy, clear or cloudy days and nights. For any given set of climatic conditions, it is acknowledged that the effect of reservoir operation on Madison River has been to reduce the total quantity of ice, to delay the formation of gorges, and to hasten the return to normal conditions. This effect results from physical causes which, if valid for one part of the valley, must be valid for all parts. What then produced the abnormal overflow in the Madison Valleys in 1916-17? The answer is readily found in abnormal climatic conditions.

The deduced mean natural flow of Madison River at Plant No. 2, situated between the Upper and Lower Valleys (Fig. 1), during the months, November to March, each winter, and the increases in flow resulting from the operation of the Hebgen Reservoir, are given in Table 3.

TABLE 3.

Winter season, November-March.	Deduced mean natural flow, in second-feet.	Additional flow from Hebgen Reservoir, in feet.	Percentage of increase.
1914-15	1 600	440	28
1915-16	1 540	520	34
1916-17	1 440	940	65
1917-18	1 420	610	43
1918-19	1 460	1 090	75
1919-20	1 170	180	15
1920-21	1 310	110	8

The Hebgen Dam is about 60 miles below the source of the river and by creating a reservoir about 20 miles in length, has the effect of eliminating approximately 100 miles of river which before the construction of the dam contributed large quantities of ice to the lower valleys. Ice gorges now form above the reservoir as in previous years, but this mobile ice is melted in its passage under the surface ice of Hebgen Reservoir. The water issuing from the dam is free from ice and its temperature is slightly above the freezing point; therefore, no ice can form in this river until the water has again been cooled to the freezing point. It has been found by observation that no appreciable quantity of frazil ice even in extreme temperatures begins to form within 10 miles below the dam.

The effect of the Hebgen Reservoir, therefore, aside from the question of increased flow, has been to diminish the quantity of ice reaching the Upper Madison Valley. For the same reasons, the Madison Reservoir which is at the head of the canyon between the two valleys (Fig. 1), has the effect of still further diminishing the quantity of ice in the Lower Valley by eliminating

about 80 miles additional of ice-producing river channel. Hence, before the construction of either reservoir, the Lower Valley was the gathering ground for the ice produced in 150 miles of river. By the construction of the reservoir the length of ice-producing channel has been reduced to about 25 miles.

During the winter of 1916-17, as a result of the operation of the Hebgen Reservoir, the mean increase in flow was 65%, and during certain periods, it was more than doubled. Yet the increased quantity, if no ice was present, is only about one-fourth the capacity of the river channel before overflow occurs.

Ice-Forming Factor

The real cause of the unusual overflow of 1916-17 in the Madison Valley was abnormally sustained moderately low temperatures. The rapidity with which ice will form is dependent far more on the drop in temperature below freezing than any other factor, provided that drop is not sufficient to form a "bridging gorge" throughout the length of the river and thus prevent the further formation of ice. Clear days and nights, cloudy weather, winds, and snows, all have their effect, but they are of minor importance in comparison with the temperature gradient.

The quantity of ice formed will depend on the amount of heat lost from the water. The heat lost after water has been cooled to 32° Fahr. and ice has begun to form is the latent heat of fusion. Virtually, 1 lb. of water is frozen for every 144 B. t. u. extracted from the water. Practically, the same amount of heat is lost in 1 day with a drop in temperature of 10° as is lost in 2 days with a drop of 5°; hence, the number of degree-days below freezing in any period will be the measure of the quantity of ice formed during that period.

When the air temperature is above the freezing point, heat passes from the air to the water, and latent heat is absorbed by the water with a reduction in the proportion of ice. The quantity of ice thus lost is also measurable by the degree-days above freezing. Hence, in any period, the net quantity of ice is measured by the excess of degree-days below freezing over those above that temperature. These relations, of course, only hold as long as the river remains open and a mixture of ice and water may be considered.

Fortunately, long-time records of temperature were available at Bozeman (and Fort Ellis), Mont. These records beginning in 1868 were practically continuous to date. A comparison of temperature records at Bozeman with those obtained in recent years at Madison Plant No. 2, at Ennis, and at Three Forks, shows that the Bozeman records represent very accurately the temperatures in both the Upper and Lower Madison Valleys, and may be safely used to fix relative yearly variations.

The mean daily temperatures for the months of November to March, inclusive, for each winter, were plotted. The winter season was considered to begin on the first date in the fall that the mean daily temperature fell below freezing and to end with the date it rose above freezing in the spring. The areas of these temperature curves below freezing in excess of those above freezing were found by means of a planimeter. These areas gave for each winter season the number of degree-days below freezing in excess of those

above freezing. This quantity was called the ice-forming factor for that season. Figs. 6 and 7 show the temperature curves for the winters of 1916-17 and 1871-72, respectively. Fig. 8 shows the ice-forming factors for each of the 53 years of record, arranged in chronological order, and Fig. 9, the same factors arranged in order of magnitude. With the exception of the winter of 1871-72, it is seen that the winter of 1916-17 shows the greatest ice-forming factor during the entire 53 years.

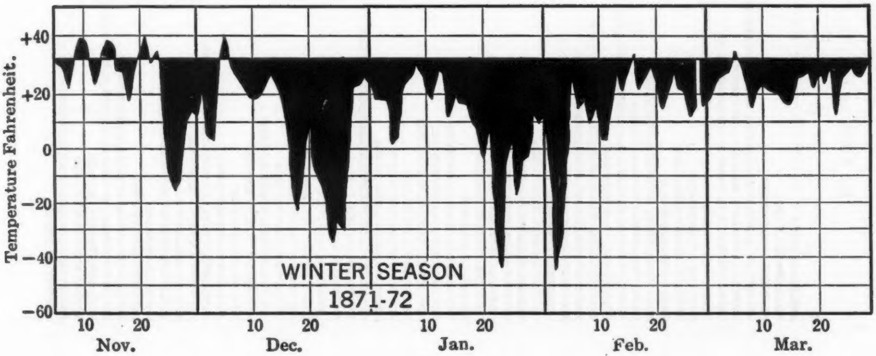

FIG. 6.

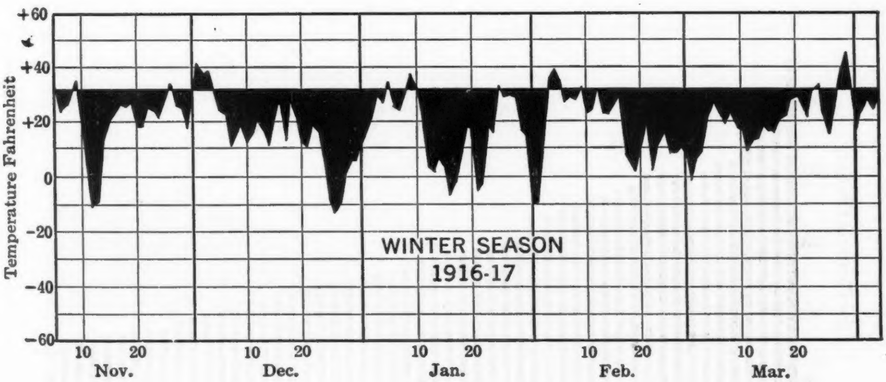

FIG. 7.

The ice-forming factor alone is no measure of the amount of overflow, but merely indicates the net quantity of ice-forming influence from temperatures that obtained during the winter. Overflow results from the type of gorge produced, and one must look to the daily temperature curves themselves in order to judge of the nature of the gorges that would be formed. A comparison between Figs. 6 and 7 shows at a glance why the gorge produced during the winter of 1916-17 was of the overflow type, while that of 1871-72 was more likely to have been a "bridging gorge" which did not produce any unusual amount of overflow.

From this analysis the conclusion was inevitable that the year showing the greatest ice-forming factor and at the same time in which the highest temperatures obtained, was the one producing maximum overflow. Of all the years of record, the winter of 1916-17 was the most abnormal in this regard.

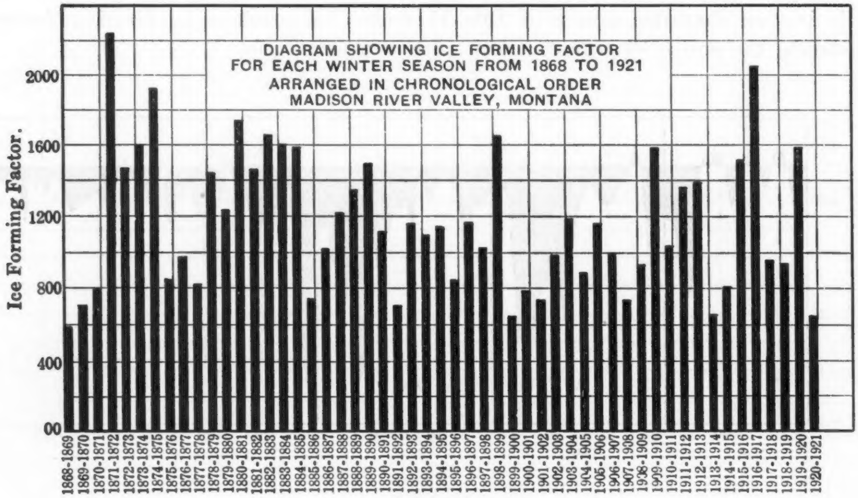

FIG. 8.

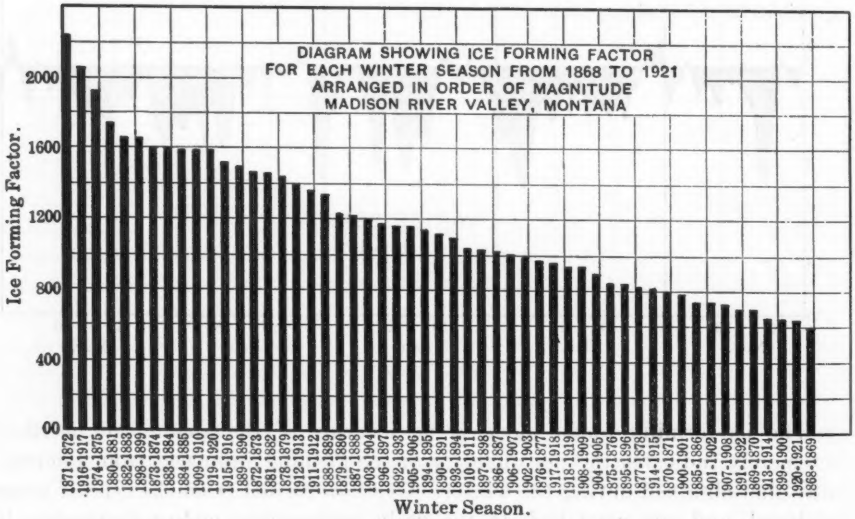

FIG. 9.

FLUCTUATIONS CAUSED BY ICE GORGING

Obviously, when an ice gorge has formed and overflow has commenced, great fluctuations in flow and in river stage are produced. During the formation of the gorge a great quantity of water is converted into ice and remains in

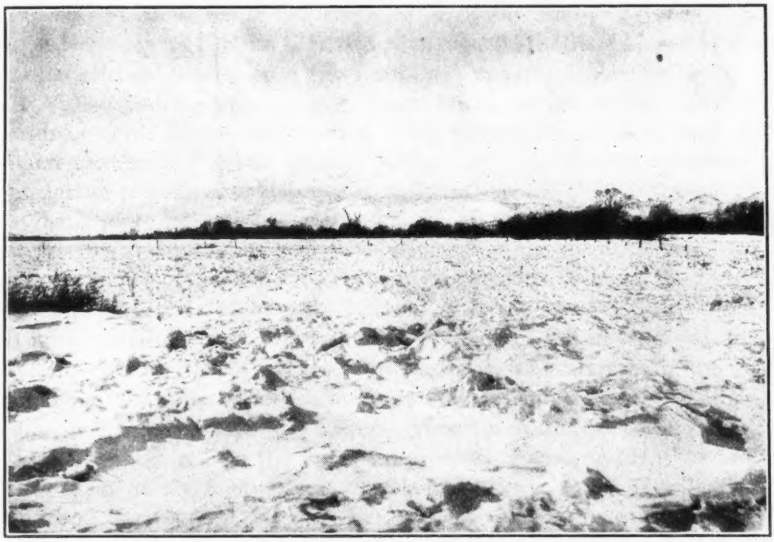

FIG. 10.—FRAZIL ICE LEFT ON AGRICULTURAL LANDS AFTER RIVER HAS RETURNED TO ITS CHANNEL.

FIG. 11.—BROKEN ICE LINE ON BANK SHOWS HEIGHT TO WHICH WATER WAS RAISED AT TIME OF FORMING OF GORGE.

temporary storage until released by a rise in temperature. In fact, it was the inability to secure water for power-plant operation in the canyon between the two valleys which led to the construction of the Madison Reservoir.

In order to secure definite data regarding the fluctuations caused by gorging, two automatic water-level recorders were used, one at the Varney Bridge in the Upper Valley and the other at the Three Forks Bridge in the Lower Valley. To insure records during the freezing temperatures, the floats of each recorder were arranged to rest on oil. In this manner water levels were secured at both places during periods of gorge formation.

At both places, the gorges formed were of the "bridging" type, with the water flowing under the ice and carrying varying quantities of frazil ice from the open river above. Once the gorge was formed, the fluctuations follow broadly the variations in temperature.

It is the general tendency for water to rise with a fall in temperature. This is explained by the slowing up of the current due to an increase in the quantity of frazil ice. A drop in temperature is also coincident with a reduction in the flow, due to the temporary increase of water stored in the form of ice in the river channels, and in the increased areas of flowing water.

One effect of the fluctuations which is damaging to meadow lands, is the cutting and the removal of sod. Evidence of this was plainly seen near the head of the Madison Reservoir and in certain areas near the head of the gorges in the Lower Valley. In some of the meadows and pastures, the sod had been badly cut by the ice. Examined early in June, 1917, the peculiar phenomena were seen of irregular patches of sod from 1 to 50 sq. yd. in area resting on top of several feet of ice, the explanation of which is simple.

The first overflow covered the land with a sheet of water, which, in freezing, became attached to the sod. A second flooding covered this ice 1 ft. or more in depth. The ice on the bottom was cracked and broken in irregular patches and rose to the surface carrying with it the sod on which it was first formed. Later, as the gorge increased, the depth of water became greater and the current flowing under the surface ice, being impregnated with frazil ice, soon filled completely the space between the surface ice and the ground. The ice under the sod is protected and is the last to disappear when the ice melts in the spring. Hence, these patches resembled huge toadstools of ice with sod caps.

Some of these ice cakes with sod adhering to them float away and in melting deposit the sod in adjacent fields. Generally, however, by becoming a part of the surface ice in the ponded water immediately above the ice gorge, they are held in place and, in melting, deposit the sod near the place from which it came, except that it is usually moved a few inches down stream or tilted and broken.

There are evidences throughout both valleys that this process is a natural phenomena of overflow from ice gorges and has been going on from time immemorial.

Remedial Measures

It was previously pointed out that the banks of the Madison River are not sufficiently high to retain the water in its channel while the stage increases as

a result of retarded velocities due to its ice load. Overflow often occurs before the surface can bridge over with ice and thus prevent the further formation of frazil and anchor ice.

The obvious remedy is to increase the height of the banks by dikes. In the Lower Valley, as a result of this investigation, a diking district was organized by the land-owners affected and a dike 11 miles in length was built. The top of the dike was constructed 10 ft. above low water, and the dike was placed as near the river bank as possible consistent with protection from under cutting. Where necessary, ordinary culverts without trap-gates were built through the dike for drainage. The effect of this will be to prevent future overflow, but it will be necessary to irrigate some of the lands which previously were watered from these overflows. There is no doubt but that the winter overflow was a benefit for the raising of wild hay crops, and whether or not all overflow should be prevented can only be determined on the merits of each tract or area as governed by its local conditions.

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PAPERS AND DISCUSSIONS

THE AREA OF WATER SURFACE AS A CONTROLLING FACTOR IN THE CONDITION OF POLLUTED HARBOR WATERS.

By Richard H. Gould,* Assoc. M. Am. Soc. C. E.

Synopsis.

The sources of oxygen required to meet the demand of the polluting matter in the waters of New York Harbor are discussed in this paper. It is shown that the air is the most important source of this oxygen which is obtained by direct absorption through the water surface. The conclusion is, that under aerobic conditions, oxygen dissolved in the harbor waters will be depleted by the demand of the polluting matter until a condition is reached where the oxygen absorbed from the air equals in quantity that abstracted by the fermenting material in the waters.

During the past year, the writer has had occasion to make a number of tests and observations on the pollution of the waters of parts of New York Harbor and some of its tributaries. As a result of this work, and of other studies, he has been impressed by the tremendous part apparently played by atmospheric oxygen in supplying the oxygen demand of the polluted harbor waters and in preventing obnoxious conditions. The importance of atmospheric oxygen in this connection has been stressed by other writers, notably by Adeney in his report to the New York Metropolitan Sewerage Commission, published in 1912. Others, however, have credited this factor with little influence on harbor conditions. In advancing opinions in support of the former view, the writer recognizes that the basic data available for study are meager, and this paper is presented with the purpose of developing additional information.

The data under discussion were secured by the writer in a series of dissolved oxygen analyses, covering a period from July to December, 1920, made in con-

Note.—Written discussion will be published in a subsequent number of *Proceedings*, and, when finally closed, the paper, with discussion in full, will be published in *Transactions*.

* New York City.

nection with a report by James H. Fuertes, M. Am. Soc. C. E., to the city authorities of Elizabeth, N. J. These data were taken principally on the Jersey side of New York Harbor, mostly in Newark Bay, Kill Van Kull, Arthur Kill,

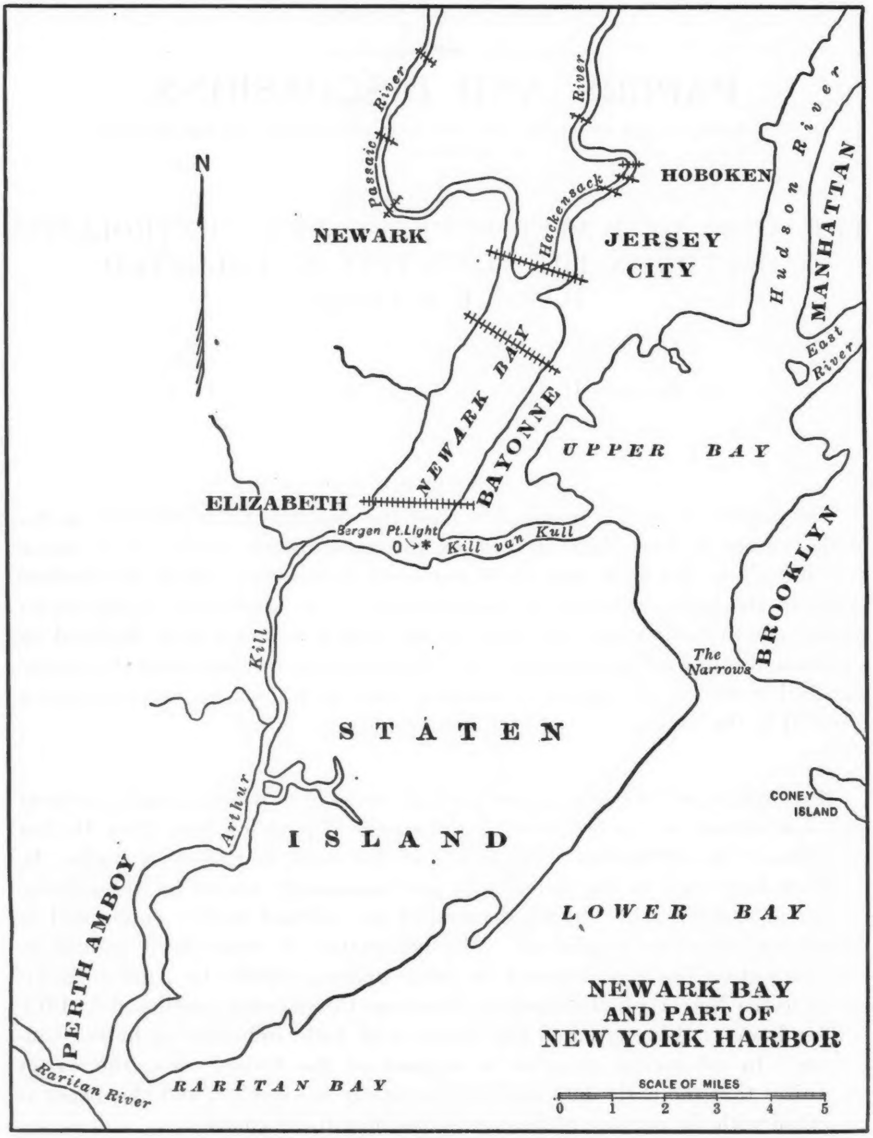


FIG. 1.

Raritan Bay and tributary streams (Fig. 1). The analyses were carried out in accordance with methods used by the Metropolitan Sewerage Commission, and the large amount of data given in the valuable reports of this Commission

have been used freely to supplement the new data which necessarily were limited.

In the course of the investigations, it was observed that at times the dissolved oxygen content of the water changed rapidly from day to day and that the changes seemed to be due primarily to the wind, that is, following a brisk wind which caused considerable wave disturbance, the dissolved oxygen content of the waters might be much higher than is usual in calm weather under similar conditions of temperature and dilution with sea water. For example, on August 24th, 1920, the dissolved oxygen in Newark Bay, at the Lehigh Valley Railroad Bridge, was 14% of saturation, and eight days later, at this point, it was found to be 75% of saturation. In the first case, there was little wind and wave disturbance, whereas in the latter there was much wave motion caused by a brisk northwest wind. At another time, in the Arthur Kill, at Elizabeth, N. J., the dissolved oxygen value increased from 19% at 9.25 A. M., to 52% at 4.00 P. M. of the same day. No doubt, these are extreme cases, but they indicate the importance of the wind and possibly other natural phenomena on the dissolved oxygen content of the harbor waters.

The fluctuation of dissolved oxygen values, in the Arthur Kill, at Elizabeth, is shown graphically on Fig. 2 for certain days in the period from July to December, 1920. The percentage of sea water present and the temperature of the water are also shown on the diagram. Average daily wind velocities, as obtained by the Weather Bureau on the roof of the Whitehall Building in Lower Manhattan, are also shown. This station is about six miles from Newark Bay and at considerable elevation, hence the velocities obtained do not represent actual wind velocities at the surface of Newark Bay. They do show, however, the relative fluctuation in the wind currents, and are valuable, at least, to that extent.

It may be noted that the majority of high saturation values follow or are concurrent with relatively high wind velocities, and, conversely, that the lower saturation values follow lower wind velocities. A quantitative relation between wind velocities and dissolved oxygen content is not apparent from present data, but such a relation could be obtained, no doubt, if sufficient data were available on wind velocities and their direction and on dissolved oxygen content. Although this information would be interesting and of some value, engineers are concerned to a greater extent in the quiescent condition of the harbor waters. The critical conditions for fish life and for the production of nuisances occur when the dissolved oxygen is depleted to the greatest extent.

The chief interest in the effect of wind velocity on the dissolved oxygen content, as far as this paper is concerned, is the attention it draws to the importance of atmospheric oxygen in the maintenance of dissolved oxygen values in the waters. It seems probable that the chief effect of the wind is to ruffle the water surface and, therefore, increase the area exposed to the air. That such decided changes in the dissolved oxygen content occur when the area of the water surface is thus increased, leads to the opinion that oxygen absorbed from the air even through the quiescent water surface may be of considerable importance. An effort was made, therefore, to define the relative importance

of the sources of oxygen to the polluted waters by a study of conditions in Newark Bay.

Newark Bay is a shallow basin, with an average depth of about 6.9 ft. at low tide and a water surface area of about 8.06 sq. miles. Its location in reference to New York Harbor is shown on Fig. 1. The Hackensack and Passaic Rivers flow into Newark Bay at its upper or northern end, and their outlet to the ocean is through two channels, namely, the Kill Van Kull, connecting Newark Bay with Upper New York Bay, and the long reach of the Arthur Kill, connecting Newark Bay with Raritan and Lower New York Bays. The waters in the tidal prisms of Newark Bay and of the Passaic and Hackensack Rivers flow back and forth through the two "Kills". According to the U. S. Coast and Geodetic Survey, 84% of these waters pass through Kill Van Kull and 16% through Arthur Kill.

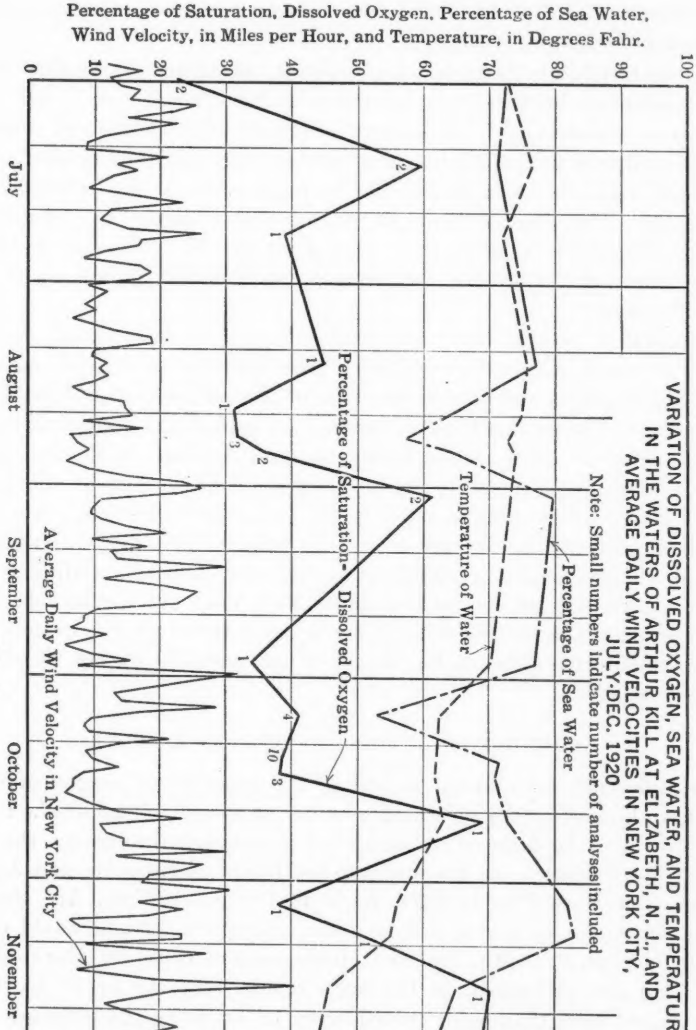
The waters of Newark Bay are heavily polluted. The inflowing Passaic River has been in a septic condition for years, since it receives the untreated sewage from Paterson, Passaic, Newark, and smaller communities. The Hackensack River is polluted to a lesser degree; however, it carries considerable waste material, and all the bordering cities, Jersey City, Bayonne, Elizabeth, and Newark, as well as the Borough of Richmond, New York City, discharge a portion of their untreated sewage into the waters of Newark Bay. The waters of Upper New York Bay are polluted by sewers draining into New York Harbor, and tidal currents force a part of these waters into and out of Newark Bay.

An estimate was made of the amount of this waste material entering the Bay and also its probable oxygen demand for the period of its retention in the Bay waters. The visible sources of oxygen to meet this demand were then measured and estimated to determine the relative importance of the various oxygen sources in preventing obnoxious conditions in the Bay.

The volume of the polluting material can be estimated approximately from the population whose wastes flow into the waters. For purposes of the computation, all the sewage flowing into the tidal waters of the Passaic and Hackensack Rivers and Newark Bay was included. The City of Elizabeth lies at the junction of Newark Bay and Arthur Kill, its sewage being carried north into Newark Bay on the flood tides and south into Arthur Kill on the ebb tides. For present purposes, one-half the volume of its sewage and that from a joint municipal sewer discharging at Elizabeth are considered as being tributary to Newark Bay.

The determination of the oxygen demand of the sewage which finds its way into the Bay is more complicated. This demand is dependent on the strength of the sewage, on the length of time it is in contact with the waters of the Bay, on the temperature, and on the nature of the bacterial actions taking place.

In a report to the Metropolitan Sewerage Commission published in 1912, Adeney estimates that the total oxygen requirement of settled New York sewage, at 0° cent., and 760 mm. barometer, would be not more than 250 cu. cm. per liter, and that about 60% of this amount would probably represent the demand during 48 hours under actual harbor conditions in summer, with



Percentage of Saturation, Dissolved Oxygen, Percentage of Sea Water,
Wind Velocity, in Miles per Hour, and Temperature, in Degrees Fahr.
0
10
20
30
40
50
60
70
80
90
100
VARIATION OF DISSOLVED OXYGEN, SEA WATER, AND TEMPERATURE
IN THE WATERS OF ARTHUR KILL AT ELIZABETH, N. J., AND
AVERAGE DAILY WIND VELOCITIES IN NEW YORK CITY,
JULY-DEC. 1920
Note: Small numbers indicate number of analyses included
Percentage of Sea Water
Temperature of Water
Percentage of Saturation- Dissolved Oxygen
Average Daily Wind Velocity in New York City
July
August
September
October
November
December

FIG. 2.

a temperature of 25° cent. This latter figure corresponds to about 570 cu. cm. of oxygen per gallon of sewage.

The Metropolitan Sewerage Commission has made a number of small-scale experiments on the oxygen demand of New York sewage, with different dilutions of sewage and fresh water, the results of which varied considerably with the dilution used. From its work, however, the Commission estimated that about 1 428 lb. of oxygen would be required to oxidize 1 000 000 gal. of raw sewage. This amount is about 80% of that estimated by Adeney for the demand of settled sewage in New York Harbor. The estimate of the Metropolitan Sewerage Commission corresponds to about 455 cu. cm. of oxygen per gallon of sewage.

It is well to bear in mind the great difficulty in reproducing harbor conditions in small incubation samples. The dilution of sewage with harbor waters is uncertain, and this is also true of the actual time during which the sewage stays within the harbor limits. An additional consideration is the probable existence of a rather strong bacterial culture in the waters of the harbor. It is quite usual to see a marked brownish floc in the waters of Newark Bay, similar in appearance to that of activated sludge. More reliable data are needed on the oxygen demand of sewage in harbor waters, but with the limited information available it seems that Adeney's estimates for the oxygen requirement of settled sewage in New York Harbor may be accepted for the present. In the following computations, therefore, the oxygen demand for sewage entering Newark Bay has been taken at 570 cu. cm. of oxygen per gallon of sewage.

Absorption of Atmospheric Oxygen by Water.

The differences of opinion regarding the quantity of oxygen absorbed by water have hinged largely on whether or not oxygen dissolved by the top layers could stream or be diffused or otherwise distributed throughout the body of water at lower depths. In his report to the Board of Estimate and Apportionment of New York City in 1911, Earle B. Phelps, Affiliate, Am. Soc. C. E., showed that if Ficks' law of diffusion was applicable, oxygen would not penetrate to any great depth, and that little oxygen could be absorbed by the water. He also estimated, in the same report, that the upper 12 ft. of the harbor waters were thoroughly mixed every 1.1 hours by the influence of tides, wind, shipping, etc. Adeney held as a result of his experiments that dissolved oxygen would stream from the top to the bottom of the liquid, maintaining a nearly uniform percentage of saturation throughout the depth of the water.

The latter view does not seem to be inconsistent with the results obtained by the Metropolitan Sewerage Commission, which indicated, as a rule, no marked and general difference in the oxygen values near the surface and those values toward the bottom. In Newark Bay, there seems to be little question but that dissolved oxygen is uniformly and rapidly mixed throughout the depth of the water. The tidal water coming in through the deep channels of the Kills is spread out like a fan over the shallow flats of the Bay. It is further mixed by the close-standing piles of the railroad trestles crossing the Bay, and by the uneven bottom caused by ship channels and obstructions.

The quantity of oxygen capable of being absorbed will depend on the temperature, salinity, and percentage of saturation of the liquid. Water devoid of dissolved oxygen when exposed to the air will absorb oxygen rapidly at first, the rate of absorption decreasing gradually to a low value when the capacity of the water has been reached. A number of analyses made by the Metropolitan Sewerage Commission, Mr. Rideal, and the writer, have been recomputed,

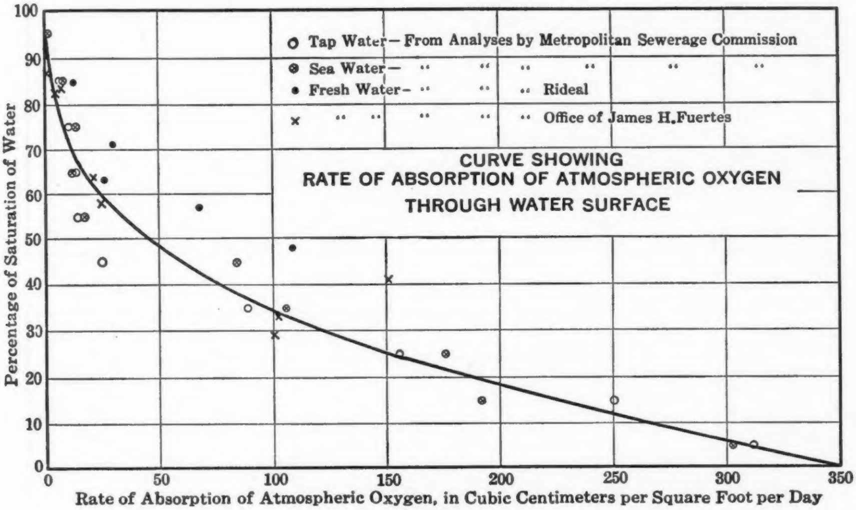


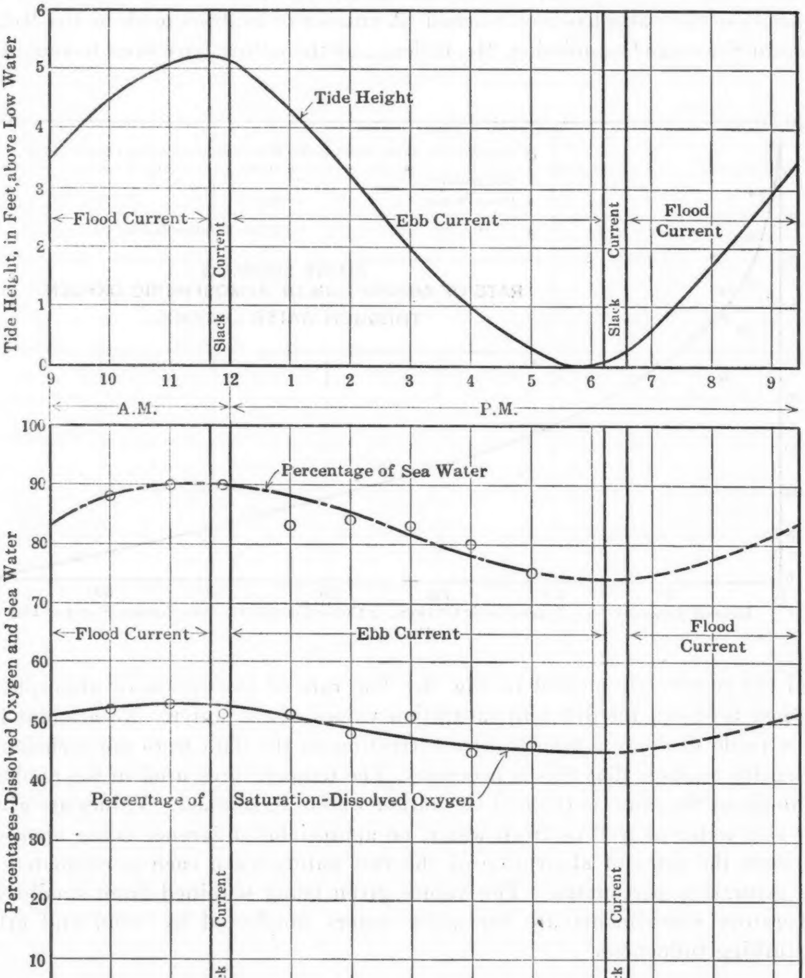
FIG. 3.

and the results are plotted in Fig. 3. The rate of absorption of atmospheric oxygen is shown for different saturation values of the water. No attempt has been made to apply a temperature correction as the data were not sufficiently extensive to show that this is necessary. The temperatures used in the analyses were about the same as those of the harbor waters in summer. Values are given for salt water as well as fresh water, no appreciable difference being apparent between the rates of absorption of the two waters when each is estimated on its saturation percentage. The values given being obtained from small-scale laboratory experiments are for quiet waters unaffected by wind and other disturbing influences.

OXYGEN SUPPLIED BY TIDAL WATERS.

One of the interesting and rather surprising facts disclosed by investigation was that the tidal waters flowing into Newark Bay through Kill Van Kull and Arthur Kill supply little oxygen for the purifying processes taking place in the Bay. It was found from an examination of the reports of the Metropolitan Sewerage Commission that from 106 analyses of flood currents and 104 analyses of ebb currents in Kill Van Kull during 1911, 1912, and 1913, the excess of oxygen in the flood currents over that in the ebb currents was only 0.10 cu. cm. per liter, or less than 1.5% of the saturation value.

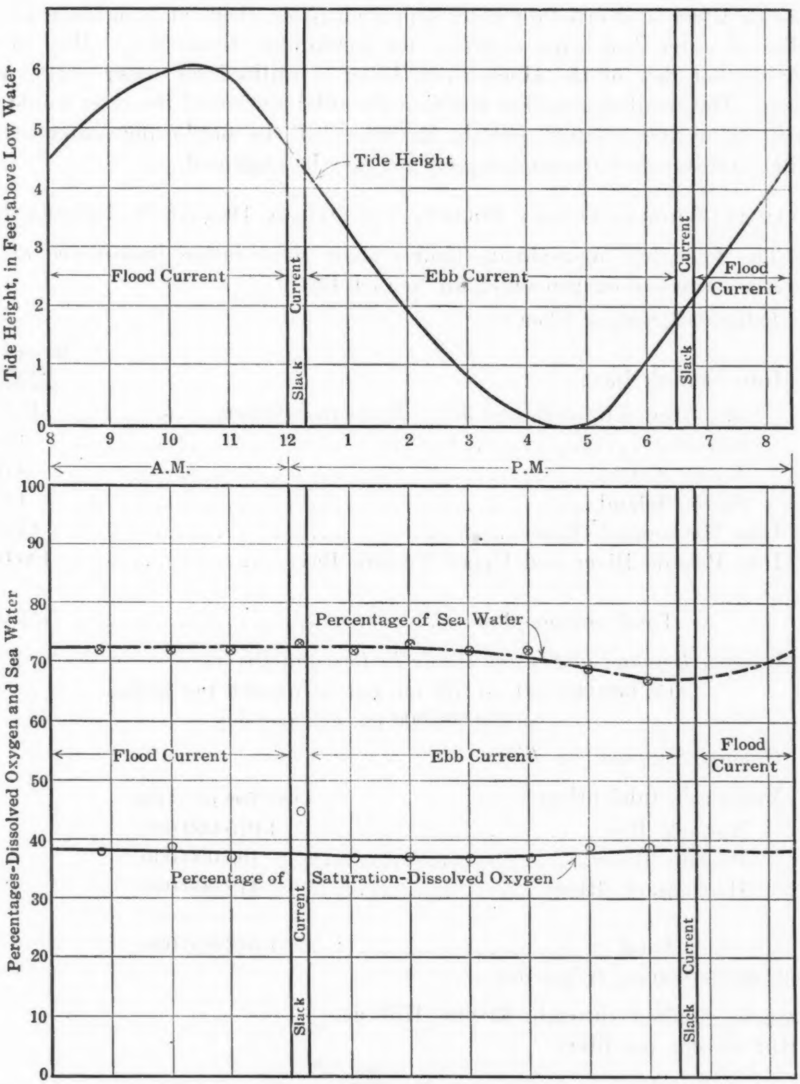
Analyses made by the writer, during 1921, covering a tidal cycle in Kill Van Kull and in Arthur Kill, at Elizabeth, are shown graphically in Figs. 4 and 5. The excess oxygen carried by flood currents over that carried by the



VARIATION IN DISSOLVED OXYGEN AND SEA WATER
IN THE WATERS OF KILL VAN KULL AT BERGEN POINT LIGHT,
OCT. 15, 1920

FIG. 4.

ebb currents was found to be 0.16 cu. cm. per liter in the case of the Kill Van Kull and 0.02 cu. cm. per liter in the case of Arthur Kill. It may be noted that the quantity of oxygen supplied to the Bay from this source is of minor importance.



VARIATION IN DISSOLVED OXYGEN AND SEA WATER
IN THE WATERS OF ARTHUR KILL AT ELIZABETH, N. J.
OCT. 13, 1920

FIG. 5.

Oxygen Supplied by Natural Stream Flow.

Periods of low saturation in the harbor waters correspond generally with low stream flows in the inflowing rivers. Part of the dry-weather flow of the Passaic River is diverted for water supply purposes, about 81%, or about 770 sq. miles of water-shed being available for furnishing stream flow. Most of the dry-weather flow of the Hackensack River is utilized for water supply purposes. The small stream flow entering the tidal section of the river would not have an oxygen content greatly in excess of the outflowing water at the Kills, and its effect in supplying oxygen may be neglected.

Example Showing Oxygen Sources and Oxygen Demand in Newark Bay.

The following approximate figures show the relative importance of the various sources of oxygen supply in Newark Bay.

Estimated Sewage Flow.—

	Millions of gallons per day.
Into Newark Bay:	
50% from Elizabeth and Joint Municipal Sewer	15.0
Bayonne	1.0
Jersey City	6.0
Staten Island	1.0
Into Hackensack River	14.0
Into Passaic River and Upper Newark Bay	120.0
Total sewage flow	157.0

Oxygen Demand of Sewage While in Bay and Rivers.—

157 000 000 gal. at 570 cu. cm. of oxygen per gallon
= 89 490 000 000 cu. cm. per day.

Oxygen Supplied by Tides.—

Volume in tidal prisms:	Cubic feet per cycles.
Newark Bay	1 070 000 000
Passaic River	195 000 000
Hackensack River	418 000 000
Total	1 683 000 000

or 3 367 000 000 cu. ft. per day.

Oxygen supplied through Arthur Kill at 0.02 cu. cm. per liter:

$$3\,367\,000\,000 \times 16\% \times 0.02 \times \frac{170}{6} = 304\,000\,000 \text{ cu. cm. per day.}$$

Oxygen supplied through Kill Van Kull at 0.16 cu. cm. per liter:

$$3\,367\,000\,000 \times 84\% \times 0.16 \times \frac{170}{6} = 12\,800\,000\,000 \text{ cu. cm. per day.}$$

Total oxygen from tides .... 13 104 000 000 cu. cm. per day.

Oxygen Supplied by Rivers.—

Passaic River.—The dry-weather flow, taken at 0.2 cu. ft. per sec. per sq. mile of water-shed is not used for water supply. The oxygen content is assumed to be 5.2 cu. cm. per liter at Paterson Falls and 1.8 cu. cm. per liter at Kill Van Kull, or a difference of 3.4 cu. cm. per liter:

770 sq. miles at 0.2 cu. ft. per sec. $\times$ 86 400 = 13 310 000 cu. ft. per day.

13 310 000 $\times$ 3.4 $\times$ 28.32 = 1 280 000 000 cu. cm. of oxygen per day.

Hackensack River.—The entire dry-weather flow is taken for water supply:

Total oxygen from rivers = 1 280 000 000 cu. cm. per day.

Oxygen Supplied from Atmosphere.—*

Passaic River.—At 0% of saturation and 40 600 000 sq. ft. of water surface:

40 600 000 at 345 cu. cm. per square foot = 14 007 000 000 cu. cm. per day.

Hackensack River.—At from 5% to 40% of saturation:

89 000 000 sq. ft. at 166 cu. cm. per square foot........................ = 14 774 000 000 cu. cm. per day.

Newark Bay.—At from 5% to 35% of saturation:

227 700 000 sq. ft. at 184 cu. cm. per square foot........................ = 41 900 000 000 cu. cm. per day.

Total oxygen from air.......... 70 681 000 000 cu. cm. per day.

Oxygen from All Sources.—

From tides........	13 104 000 000	cu. cm. per day	=	15.4% of total
From rivers.......	1 280 000 000	" " " "	=	1.5% " "
From atmosphere.	70 681 000 000	" " " "	=	83.1% " "
Total	85 065 000 000	cu. cm. per day	=	100 %

The estimated oxygen demand of the entering sewage was 89 490 000 000 cu. cm. per day, 95% of which is accounted for in the apparent oxygen sources previously mentioned. The agreement is closer than might be expected from the extent of the available data. It will be noted that 83.1% of the apparent oxygen supply is estimated as coming from the air.

An attempt was made to apply this method of computation to the entire New York Harbor. Complete data were not available, and it was necessary to make a number of approximations. Table 1 is given not because it is believed to be accurate in all its details, but because it throws an interesting light on the relative importance of the purifying agencies acting on the harbor waters.

It is recognized, of course, that the source of oxygen dissolved in water is from the air, although agreement is not general as to the rate at which this

* Absorption values are taken from Fig. 3.

takes place. Passing out from the harbor to the ocean, organic matter is diluted with such large volumes of water and spread over such a large area that the depletion in oxygen is not noticeable. The distinction between oxygen supplied from the tides and that from the air is relative only, depending on the areas considered. If the included area is taken far enough from the sources of pollution, the effects of oxygen carried by the tides are of little relative importance, as all oxygen consumed by the waters in the area is made up by absorption within the limits chosen. The area available to absorb oxygen and maintain dissolved oxygen in the waters near the sources of pollution depends chiefly on the distance and area reached by direct tidal currents and, therefore, on the volume of tidal flow passing these points. The intermixture of the direct tidal currents with waters in the areas immediately beyond their reach makes additional absorption areas available. These latter areas are polluted to a lesser degree, and therefore the unit rate of oxygen absorption is less. The extent of these areas will depend on the characters of the tidal currents, the tidal over-run, and other conditions difficult to define.

TABLE 1.—ABSORPTION OF ATMOSPHERIC OXYGEN IN NEW YORK HARBOR IN 1920.

Section.	Area of water surface, in million square feet.	Estimated minimum percentage of saturation of dissolved oxygen, in 1920.	OXYGEN ABSORBED FROM AIR.	
			Cubic centimeters per square foot per day.	Million cubic centimeters per day.
Upper Bay	542	25	152	82 500
Hudson River (to Mt. St. Vincent)	404	15	220	89 000
Upper East River	258	15	220	56 800
Lower East River	118	5	305	36 000
Harlem River	21.4	0	347	7 420
Kill Van Kull	28.7	30	124	3 560
Newark Bay	224	15	220	49 400
Arthur Kill	116	45	59	6 830
Jamaica Bay	582	45	59	34 400
Lower Bay	3 420	80	6	20 600
Hackensack River	89.1	15	220	19 600
Passaic River	40.6	0	347	14 100
Long Island Sound, Throggs Neck-Pelham	489	80	6	2 940
Total apparent oxygen absorbed from atmosphere	....	....	....	423 150

In the case of New York Harbor, the waters of the Lower Bay are still well saturated with oxygen, indicating that beyond this point oxygen is absorbed at a low rate, presumably over large ocean areas. Table 1 indicates to what extent the oxygen that may be absorbed by waters, depleted to the extent of the harbor waters, compares with the estimated oxygen demand of the sewage from the Metropolitan District. The minimum saturation values found in the harbor waters are used, as it is believed that they comply with the basis of the absorption curve more than average values which include high figures due to the disturbing influence of winds, etc. Most of the saturation values

used are estimates based on the minimum values published in the 1917 Report of the Board of Estimate and Apportionment of New York City. The absorption values of atmospheric oxygen are taken from Fig. 3.

The oxygen content of the fresh water of the rivers flowing into the harbor is not materially different from that of the waters of Long Island Sound and the ocean at Sandy Hook, so that the influence of the oxygen carried by them may be neglected. If the total sewage flow of the Metropolitan District is taken as 800 000 000 gal. per day, the 423 150 cu. cm. of oxygen per day given in Table 1, represents 529 cu. cm. of atmospheric oxygen per gallon of sewage utilized by the organic matter in the sewage while it is within the harbor limits. This is about 16% greater than the estimate of the Metropolitan Sewerage Commission for the oxygen demand of New York sewage and about 7% less than that of Adeney as given previously. This seems to indicate that nearly the entire oxygen demand of the sewage in the harbor waters can be taken care of by direct absorption from the air. From this, it seems reasonable to conclude that the percentage of saturation of the harbor waters at any point will be lowered to such a value that the oxygen absorptive powers of the waters at this percentage of saturation will equal the oxygen demand of the fermenting organic matter in these waters.

A similar computation was carried out for the tidal section of the Elizabeth River. This is a small, badly polluted stream flowing into Arthur Kill. It has a water surface of 1 028 000 sq. ft., and during the four tests (Table 2), the stream flow ranged from 1.6 to 10 million gallons per day and the percentage of saturation of dissolved oxygen from 12 to 32 per cent. The polluting sewage was estimated at 800 000 gal. per day. Table 2 shows the results of the four tests.

TABLE 2.—Sources of Dissolved Oxygen in the Elizabeth River.

Date, 1920.	Oxygen Supply, in Million Cubic Centimeters per Day, from:				Percentage of estimated oxygen demand.
	Stream flow.	Tidal flow from Arthur Kill.	Absorbed from atmosphere.	Total.	
June 30.........	30.2	158	267	455	100
July 29.........	17.6	240	267	524.6	115
August 12......	131.0	146	123	400	88
October 22.....	50.0	293	227	570	125
Average........	....	....	....	487.4	107

Although there is some variation, due possibly to inaccuracies in the various estimates made, the results in general tend to confirm the applicability of the method of computation.

To sum up the questions discussed, the evidence seems to indicate that the oxygen demand of polluted harbor waters, when not supplied from other sources, is satisfied as far as possible by direct absorption from the air and that the dissolved oxygen in the waters will be depleted until the rate of oxygen absorption from the air equals the rate of oxygen demand of the fermenting organic

matter; that is, the area of the water surface available to absorb oxygen from the air appears to be the principal factor in controlling the condition of polluted harbor waters. The volume of water in the harbor and tidal currents, although of tremendous importance in mixing the waters and distributing the polluting material over large areas, seems to be secondary to the ability of the waters to absorb oxygen directly from the air.

Data on the oxygen demand of sewage in the waters of New York Harbor and for the construction of an absorption curve are not as complete as might be desired. It is believed, however, that the values given by the writer are sufficiently near the actual conditions to bear out at least the underlying ideas presented. With more extensive data it might be possible that the methods indicated will permit a reasonably accurate estimate to be made of limiting quantities of sewage which the harbor will receive without the production of an active nuisance, or to predict the extent of depletion in the dissolved oxygen content for definite quantities of polluting material.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

STREAM POLLUTION AND SEWAGE DISPOSAL*

A SYMPOSIUM

By Messrs. George T. Hammond, Kenneth Allen, John F. Skinner, W. L. Stevenson, Earle B. Phelps, T. Chalkley Hatton, Langdon Pearse, and W. H. Dittoe.

With Discussion by Messrs. Harrison P. Eddy, J. Frederick Jackson, F. A. Dallyn, W. F. Wells, Alexander Potter, Edward S. Rankin, and S. John Scacciaferro.

* Presented at the meeting of November 16th, 1921.

TANKS AND FINE SCREENS FOR TREATING SEWAGE

By George T. Hammond,* M. Am. Soc. C. E.

In considering the title of this symposium, "Stream Pollution and Sewage Disposal", and what should be said about tanks and fine screens, the speaker's first thought was that perhaps the title should have been "Sewage Disposal and Stream Pollution", for observation of actual conditions rather tends to justify the statement often made, that sewage disposal is the main cause of stream pollution; this is sometimes the case even where treatment plants have been established. Not to mention instances known to most engineers, in which sewage is "disposed of" without any attempt at treatment, examples of American idealism and carelessness in sanitary works are sometimes found, in which elaborate and costly plants are constructed, and do not function, because of ignorance or neglect on the part of those whose duty it is to care for their proper operation—sanitary engineering efforts conceived in faith, but which will not function on faith alone.

Engineers are familiar with the various forms of tanks of which quite a number are obsolete, and of fine screens that have been in use for many years in sewage treatment works, some of which are also obsolete. There has been an evolution from the ancient and odorous cesspool to the sedimentation and separate sludge digestion tanks of the present. Experience and experiments go far to prove that the latter afford the most satisfactory method of tank treatment, securing a high removal of suspended solids and complete digestion of the sludge, without much danger of a nuisance. Much experience with the design and operation, as well with overloading and abuse, of such tanks was gained during the World War, and the interested student can inform himself of many data from several important papers since published, of which the speaker will only mention the one presented by Leonard S. Doten, M. Am. Soc. C. E., to the Society.†

The speaker will not attempt any discussion of the various forms of tanks and fine screens so fully described in textbooks and so frequently seen in treatment plants abroad, but will confine himself to those which seem to have demonstrated their usefulness in ordinary American practice.

One of the latest extensive projects in sewage treatment is the proposed enlargement of the Baltimore plant, concerning which an interesting paper‡ has appeared recently. The following statement is quoted from that paper:

> "Everything points to the adoption of sedimentation tanks in conjunction with sludge digestion tanks as the most logical solution of the problem for Baltimore conditions. None of the many troubles that have to be confronted in the operation of Imhoff tanks is found in treating sewage with sedimentation and sludge digestion tanks. No foaming is met with; no time is spent squeegeeing; no scum has to be removed; no skimming of tanks is required; and no uncertainties of operation have to be considered."

The experimental work on sewage treatment in Brooklyn, extending over more than five years, appears to justify the conclusion reached in Baltimore

* Brooklyn, N. Y.

† *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1918), p. 337.

‡ *Engineering News-Record*, Vol. 87, p. 654.

as to the success of sedimentation and separate digestion, except that much difficulty was not found with the Imhoff tank.

The Imhoff tank affords a method of obtaining separate sludge digestion in a chamber placed under the sedimentation chamber. It eliminates the necessity of transferring the settled matter from one tank to another, and thus has an advantage over the separate tank system. That it has some unpleasant tendencies, however, must be admitted.

In the experimental work in Brooklyn, three Imhoff tanks were used, designated, respectively, as No. 1, No. 2, and No. 3. The capacities per day were as follows: No. 1, 150 000 gal.; No. 2, 100 000 gal.; and No. 3, 50 000 gal., each operating on a theoretic 2-hour retention period. The tanks were of different depths, but of the same plan, the cross-section at any proportional depth being identical in all of them. All the tanks were operated more than five years.

Among the points of interest observed during the Brooklyn experiments, the speaker will mention the following:

1.—For Imhoff tanks the sludge digestion capacity should be sufficient to carry the sludge for eight months without discharging, and the capacity below the slots should be at least about 2 cu. ft. per capita for the population served. This large storage capacity not only is necessary for storage during non-drying periods, but it also appears to have some important relation to the digestive process, and helps to prevent foaming.

2.—Double slots, that is, one slot at the bottom of each inclined plane of the sedimentation chamber, with a triangular baffle beneath, instead of one plane passing under the other, appear to give the best results in these tanks and largely eliminate the need of squeegeeing the slopes.

3.—The slopes should be at least $1\frac{1}{4}$ vertical to 1 horizontal, and the flow should be parallel with the slopes and slots; a rate of flow equal to 1 ft. per min. is excellent.

4.—Scumboards and baffles add to tank efficiency.

5.—From observation the speaker has reached the conclusion that two sludge digestion pockets under the sedimentation chambers are better than three, and that the best length for the sedimentation chamber would afford 60 ft. net for the flow between the scumboards at the inlet and the outlet. Provision should be made to reverse the flow.

During the World War, and while in the employ of the U. S. Government, among various assignments, the speaker was detailed to take care of the design of an Imhoff tank plant for Harriman Village and other industrial housing projects, and the City of Bristol, Pa., jointly. W. H. Boardman, Assoc. M. Am. Soc. C. E., of Philadelphia, Pa., was retained by the City of Bristol, and collaborated with the speaker on this work, which consisted of three Imhoff tank units, each of which had a capacity to care for 2 500 000 gal. per day of sewage flow, at 1-hour retention, or 1 125 000 gal. per day at 2 hours. The principles previously mentioned were carried out in the design of the plant, which has been in operation about three years and has given excellent service. There has been no trouble with the plant, no odors, and squeegeeing of the slopes is seldom required.

Many engineers have come to the conclusion that in a treatment plant the Imhoff tank is an element of risk. After having visited most of the

larger and many of the smaller installations of this tank in the United States and abroad, the speaker's opinion is that to a considerable extent its bad name is due to faulty design, and, still more, to faulty operation.

Relative to the retention period in tanks: Experience and experimental work indicate that this period should be short, usually not more than 2 hours, even 1 hour will give the best results in many cases. Sedimentation is very rapid during the first hour, septic action may intervene during the second hour, and it is always desirable to keep the liquid part of the sewage as fresh as possible. Experimental results as well as observations tend to show that although not affording the high percentage of removal obtained by longer periods, a period of 2 hours gives all the preparation required for sprinkling filters or for direct discharge into a waterway affording sufficient dilution.

Since tanks of all kinds have always given some trouble, even if such trouble is merely psychological, which must be taken into account, a method of mechanical removal of suspensa from sewage has long been sought. The fine screen is the principal result of this search and the advantages secured by apparatus of this type are many and obvious. Fine screens act on the sewage at once as it enters the plant, preserving its freshness, and do not foul the night flow. They may be used as the sole method of treatment where the effluent is discharged into a waterway which at all times has sufficient volume to provide the necessary dissolved oxygen to receive it. The same remark applies also to tank effluents of all kinds. Neither the fine screen nor the tank provides a complete treatment, and, in many cases, the use of either method must be followed by filtration. The tank will remove more suspensa from the sewage than the screen.

The problem of sewage treatment is mainly the separation of solids from liquids. Ordinarily, fine screens may be said to remove from 12% to about 30%—even higher in some cases—of the settleable solids. This quantity is considerably less than is removed by the tank, but the greater freshness of the effluent from the screen, and its freedom from probable nuisance in most cases, more than offsets the difference. Either tanks or fine screens will prepare sewage satisfactorily for application to sprinkling and other filters, as was shown by the Brooklyn experiments; therefore, either can be constructed as the advance guard of a future filtration plant.

There are many fine screen plants in operation in this country, which are giving satisfaction as the sole method of treatment. One such plant may be seen at the foot of Dyckman Street in New York City, and there is another in Brooklyn of the same type, on which observations and experiments extending over four years have been made. More such screen plants are proposed and will probably be soon built, as they appear to give all the treatment conditions require for the present and for a long time in the future.

Attention should be called to the need of providing grit catchers and coarse screens ahead of the treatment plants; also, the need of tide-gates and submerged outfalls, where necessary.

Other forms of tanks, not mentioned, are not at this time of importance in this connection. The old septic tank still has its use in some places, and is well known to the sanitarian as a friend, or as a foe. Some special forms

of tanks are of interest, such as the Travis tank, the Kreamer tank, the Dortmund tank, etc., but the results of their use have not proved them the best for stream protection, and none of them has met with much success under American conditions. The various forms of chemical precipitation tanks are being abandoned everywhere as unsatisfactory.

Where an effluent from a tank, or from a screen, is good enough under local conditions to be discharged into a waterway, it is important that it should be as fresh as possible, in order to insure against local nuisance, and that it should contain a minimum of bank-forming material which usually can be removed almost completely by a tank or with a screen of proper fineness.

The finely divided flocculent matter and colloid particles in suspension will make considerable inroads on the dissolved oxygen present, but if this is of sufficient volume and the floc remains in suspension, little, if any, harm need be anticipated. The solution of the problem depends on the oxygen demand created by the biological food supply present, and, in every case, local conditions should be studied by a competent biologist before the plant is designed.

Although separate sludge digestion tanks are now coming into considerable prominence, thus far there has not been much uniformity in their design, and much study and experiment are desirable to develop this form of tank. In some cases, duplicate settling tanks have been provided, as in some of the Doten tanks, which are allowed to fill with sediment and sludge to the point of interference, and are then laid off for digestion, the other tank being used for sedimentation; and this method has met with success. Alvord has perfected a very interesting combination of settling and digesting tanks, but the field for improvement is still very wide in regard to this part of the subject.

In closing, attention may be drawn to the fact that many large cities which are not yet treating their sewage, are situated on waterways which for many years would afford ample supplies for disposal by dilution of a tank or fine screen effluent without further treatment, but which if contamination is permitted to go forward, will soon require far more costly forms of sewage treatment.

THE POLLUTION OF TIDAL HARBORS BY SEWAGE WITH ESPECIAL REFERENCE TO NEW YORK HARBOR.

By Kenneth Allen,* M. Am. Soc. C. E.

Whether the waters are tidal or those of inland lakes, the fundamental considerations regarding harbor pollution are the same, but the effects are accentuated in sea-coast harbors for several reasons:

1.—The oscillations of the tidal flow alternate with periods of slack-water when solids settle rapidly to the bottom and form sludge banks.

2.—The presence of salt in the water tends to precipitate the soaps in the sewage, making the effluent noticeable by its milky appearance and promoting further sedimentation.

3.—Owing to the greater specific gravity of sea water, the warmer sewage rises rapidly to the top and spreads out in a thin layer, diminishing to a film of sleek which covers a large and readily distinguished area on the surface.

4.—As sludge deposits decompose, sulphureted hydrogen is formed in greater abundance in salt water than in fresh water, owing to the breaking down of the sulphates contained in the sea water. Sulphureted hydrogen is known as the most characteristic of the offensive odors of putrefaction.

In experiments made for the Metropolitan Sewerage Commission, the effect of salinity on buoyancy was well shown by releasing varnished croquet balls, weighted to a specific gravity of 1.000, at a given depth and noting the time of ascent. In a mixture containing 28% of sea water (specific gravity 1.007), the upward velocity was 3 in. per sec., while with a mixture containing 86% of sea water (specific gravity, 1.0215), this velocity was doubled.

The ascent of sewage from a submerged outlet would always be less than that of a solid ball, due to rapid diffusion forming a mixture constantly approximating in character the water of the stream. Therefore, the velocities mentioned may be taken as indicating the limiting maxima for sewage discharged in still water.

For the reasons stated, the discharge of sewage into salt water is very likely to be objectionable, particularly if it is discharged in a septic state, and under otherwise similar conditions, more care is required for its proper disposal than in fresh water.

An excellent illustration is found in the discharge of nearly 100 000 000 gal. per day of septic sewage within a period of about 2 hours, about high tide, at Moon Island in Boston Harbor. This spreads rapidly over the surface so that several hundred acres of water has the appearance of undiluted sewage, although samples taken by the Metropolitan Sewerage Commission in 1911 indicated an abundance of dissolved oxygen at a depth of 1 ft. or more. In fresh water, diffusion would be much more general.

In the Borough of Manhattan, which to many is synonymous with New York City, the untreated sewage passes from about 180 outlets directly to the salt water of the surrounding streams. The result is that deposits form in the slips and under the piers so that dredging is necessary, and conditions offensive to the eye and nose result. This is also true of much of the Brooklyn and Bronx

* New York City.

water-fronts, but conditions are better in the other less densely populated boroughs.

These unfortunate results of the present method of disposal could be largely avoided by the general introduction of fine screening, which is already being done at Dyckman Street, Manhattan, Hendrix Street, Brooklyn, and at the 43d Street, Oak Street, and Thirty-second Avenue outlets in the Borough of Queens. There would still be left in the water, however, a large part of the impurities in solution or in a finely divided state, making a continuous demand on the dissolved oxygen.

From 1909 to 1914 the depletion thus caused has been determined by the Metropolitan Sewerage Commission, Dr. G. A. Soper, M. Am. Soc. C. E., President, and since then, by the Board of Estimate and Apportionment, New York City, Nelson P. Lewis, M. Am. Soc. C. E., and, more recently, Arthur S. Tuttle, M. Am. Soc. C. E., Chief Engineer.

The warm-weather record is interesting, as saturations are always high in winter. The most salient features are:

1.—The annual occurrence of total depletion for short periods in the Harlem River.

2.—The rapid decline in saturation in the Lower East River, reaching zero for the first time in September, 1921.

3.—The continued and general lowering of saturation in all the other main branches of the harbor, particularly at the Narrows, where a resultant of the depletion that has taken place in the Bay and rivers above may be noted.

It has been supposed that, although odors are not evolved as long as any supply of oxygen remains, they are likely to occur locally when channel saturations are less than 20 or 30 per cent.

Investigations have failed to support this view, as far as New York City is concerned, for during the past season a large part of the harbor has held less than these percentages of oxygen, without any resulting nuisance. Fish life, however, is undoubtedly interfered with, since it has been sufficiently well demonstrated that most edible varieties will not thrive where the percentage of saturation is under 30. The remedy for this phase of pollution lies in some form of tank treatment, by which most of the fine solids and some of the colloidal matter may be removed.

Large areas of the harbor covered with a film of oil may be mentioned as having caused much complaint recently. This condition has been prevalent along the Staten Island beaches and in the Lower East River, where it frequently extends practically from shore to shore. The cause is due in part to extensive oil works on Newtown Creek and at Bayonne, N. J., but more especially by the discharge of bilge water from incoming oil-burning steamers. It has injured the bathing beaches, increased the fire hazards, and interfered with fish life.

The Committee on Rivers and Harbors of the House of Representatives has been appealed to for relief, and the Commissioner of Docks of New York City has asked for an appropriation to provide barges to receive such refuse and salvage the oil, making other disposition unlawful. Action in both cases is pending.

To summarize: The City of New York is discharging about 750 000 000 gal. of sewage daily into the harbor, mostly without any treatment. With the large tidal inflow of 12 700 000 000 cu. ft. of sea water at the Narrows and 4 700 000 000 cu. ft. from Long Island Sound twice a day, added to an average flow of upland water from the Hudson River of about 2 070 000 000 cu. ft. per day, conditions such as obtained formerly at London, Glasgow, Naples, or Havana are not likely to occur soon here; but the rapid reduction of dissolved oxygen in the harbor is a danger signal which should not go unheeded.

Several important steps have been taken already to improve conditions: A number of intercepting sewers are, or soon will be, under contract, and within two or three years a number of modern fine-screening plants will be in operation. It is also hoped that a way will be found to provide a more thorough method of treatment for some of the more important outlets which are responsible for the polluted condition of the East and Harlem Rivers.

TREATMENT OF STORM-WATER

BY JOHN F. SKINNER*, M. AM. SOC. C. E.

The sources of stream pollution commonly considered have been sewage and trade wastes. The discharge from storm-water sewers and from storm overflows on combined sewers is another element which in some cases may merit attention. Especially is this true when the stream is small, when it flows through built-up territory or a park, or when it is used for fishing or bathing, or the water is used for drinking by cattle or by man.

It has generally been considered that before sewage enters a stream it should be treated to remove:

(*a*) Heavy solids, grit;
(*b*) Floating and coarse suspended solids, screenings;
(*c*) Half or more of the finer suspended solids, clarification;

and if the stream is small and sluggish so that insufficient dilution occurs:

(*d*) Oxidation of the effluent,

in order to avoid nuisance.

Economy usually dictates that the sewage treatment plant on a combined system shall receive only a part of the storm flow, while the remainder is discharged raw into the stream either at the plant or, as directly as possible, at convenient points along the main intercepting sewer, on the theory that, in time of storm flow, the dilution will be ample.

This is doubtless true if the stream is swollen at the time, for either the dilution is so great or such a load of other solids is washed in from the water-shed that the accession from the sewer is inappreciable.

However, if the storm-water discharge is due to a local shower, and the run-off from the water-shed of the stream is not generally augmented, the dilution may be reduced and the velocity may be so small that deposits will occur, the heavy material forming bars and the floating portions becoming entangled in the vegetation and littering the shores.

After a period of drought, when a sudden shower causes storm-water to debouch into the stream, the first flush carries the accumulated deposits from the sewers, accompanied by street refuse, followed by a greater dilution of rain water.

The discharge of storm-water being occasional, the finer suspended solids often may be oxidized without serious offense, or be carried on by the current if they are not entangled in other grosser deposit-forming materials. The treatment, indicated for storm-water, therefore, is the removal of this coarse and heavy detritus before the effluent is admitted to the stream.

The heavy solids consist of grit, ashes, sand, gravel, bones, fruit pits, tin cans, rubber, leather, and objects in part made of metal, which deposits will settle in a current of 1 ft. per sec. If the larger objects are excluded by racks, the resulting deposit will be mainly grit, ashes, sand, and gravel.

The floating solids consist of wood, straw, leaves, fruit rinds, remains of fireworks, rubber and baseballs, and toys lost in the surface sewers. If racks

* Rochester, N. Y.

are provided they will accumulate, in addition to the last mentioned, a collection of the coarse heavy objects mentioned previously.

The material to be removed, therefore, may be classified as grit and screenings. Coarse racks with bars from 3 to 4 in. apart will generally precede the grit chambers and the finer screenings will be removed later.

If the proposed detritus plant is at or near a sewage disposal plant where attendance is constant, racks may be raked at any time and will occupy the least space if placed vertical or slightly inclined. If the construction is in an isolated location where only periodic attendance is contemplated, the racks should be set on a flat incline, 3 or 4 horizontal to 1 vertical, and a large area should be provided so that the accumulation from a single discharge will not completely blind the rack and form a dam.

A velocity of about 1 ft. per sec. for a period of from 1 to 2 min. is the best for removing the grit. Heavy material settled in this way should not average more than 5% organic matter and, ordinarily, will not be offensive.

Grit chambers 10 or 12 ft. wide, arranged in parallel channels, will be convenient and, if desired, may be operated automatically by floats designed to cut in or out a sufficient number of channels to care for the flow at approximately the desired velocity. The grit may be excavated by a clam-shell bucket, removed in trucks or cars, and used for filling.

The finer screenings may be handled in various ways. When at or near a sewage disposal plant, mechanical screens may be used. At the lower end of a grit chamber, 10 or 12 ft. wide, carrying about 40 000 000 gal. per day, a 12-ft. R.-W. screen, or a 12-ft. Dorrco screen 10 ft. long may be used if properly housed and provided with power.

Another method would be to construct the grit chamber or basin with the upper part of the side next to the stream of open permeable material, so that water may pass through it leaving the screenings deposited on its surface. Maintaining a proper velocity through the chamber and keeping the porous bank from clogging would be the chief difficulties with this design.

Another suggestion which has considerable promise, is a grit chamber preceded by coarse racks and provided at intervals of about 25 ft. with several sets of inclined scumboards continued downward with inclined racks, progressively finer down stream, each of which is provided with raking platforms above and short submerged platforms projecting up stream from their lower ends to retain the screenings. These racks will not reach the bottom of the chamber, but will stop about 4 ft. above it. The down-stream end of the chamber will consist of an overflow weir which will maintain the water in the channel at the desired elevation. A drain valve for re-watering will also be provided.

All such devices will require periodic attention and cleaning and cannot be operated satisfactorily except on a methodical schedule, directed by a responsible man.

As previously mentioned, structures for treating storm-water may be made up conveniently of units about 10 or 12 ft. wide by a maximum of 10 ft. deep and from 60 ft. to 120 ft. long, designed for a velocity of 1 ft. per sec. The number of such units will depend on the quantity of storm-water antici-

pated, assuming the capacity of each channel to be 60 cu. ft. per sec. Either there must be a sufficient number of such units to carry the estimated maximum flow, or a smaller number may be constructed and a by-pass provided, as economy may dictate.

The simplest way of accommodating an extreme flow would be to construct the walls of the channels with considerable freeboard, in order that an increased head on the outlet weir would accommodate the increased discharge.

As an example of about what would be required, let us assume a water-shed of 1 000 acres, 1 mile wide and nearly 2 miles long, with the longest sewer 12 000 ft. in length from source to outfall, and with grades such that when running full, the velocity will average 5 ft. per sec. It will require, therefore, 40 min. for water to travel down the entire system, and if a period of 5 min. is allowed for the water to reach the sewer, a shower of 45 min. duration will be the shortest in which the entire water-shed will contribute and precipitation from the remotest corner of the territory reach the outfall at the same time as from every other part. If from an examination of the rainfall records of the locality, we select the 45-min. shower of such intensity that it occurs every year or two, it may be used as a basis for the computation of the run-off to be provided for at the outfall.

In Rochester, N. Y., this 45-min. shower has a rate of 1.22 in. per hour. In the residence section about 25% of the precipitation from these violent showers reaches the sewers during the period. A discharge of $1\,000 \times 1.22 \times 0.25 = 305$ cu. ft. per sec., therefore, can be computed which will require five of the grit-chamber units mentioned. Such a plant will take the discharge of a 9-ft. sewer at a grade of 1 in 1 000, or a 7-ft. sewer at a grade of 1 in 280.

This may be roughly stated as three of the previously mentioned detritus chamber units per square mile of tributary territory, subject, however, to wide variation, depending on the rainfall, the slope, and the character of the territory. A similar approximation would place the cost of construction of such a detritus plant at from $10 000 to $12 000 per sq. mile of territory.

It is appreciated that to date little has been done in storm-water treatment and that the health authorities have been interested chiefly in the protection of streams from other pollution, but the time may be anticipated when the concentrated storm-water discharge of cities will demand attention.

POLICIES OF THE ENGINEERING DIVISION OF THE PENNSYLVANIA DEPARTMENT OF HEALTH AS TO PUBLIC SEWERAGE.

By W. L. Stevenson,* M. Am. Soc. C. E.

In 1905, the Legislature of Pennsylvania enacted the "Purity of Waters Act" for the protection of the public health by controlling public water supplies and by prohibiting the discharge of sewage into the waters of the State.

The Act, however, provides that, in the case of municipally owned sewers, the Commissioner of Health, with the unanimous agreement of the Governor and the Attorney General, may permit the discharge of sewage subject to such conditions as he deems will subserve the general interests of the public health.

The evident intention of this prohibition and provision for discharge under certain conditions is to provide for varying degrees of sewage treatment, so as to maintain the streams in a reasonably clean condition and to make it practically and economically possible to use certain waters of the State as sources of public water supply and for other purposes requiring a hygienic standard.

Streams are the natural drainage channels for rain water flowing over the surface of the land and conveyed to them by storm-water conduits from the towns on their water-sheds.

It follows, therefore, even if all the sewage was excluded or completely purified, that no surface water from a populated water-shed would be fit for use as a public water supply without some form of purification to eliminate the natural contamination of the surface stream.

It is possible to produce a clean, colorless, and bacteriologically safe water supply from a grossly polluted and contaminated source. It is also possible to purify sewage to such an extent that it will be freed from all its polluting and contaminating constituents; but both procedures are exceedingly costly and often unreliable. Therefore, the wise and just administration of the "Purity of Waters Act" should be based on an economic consideration of the uses and conditions of the various waters of the State so as to secure the greatest protection to the public health by the least expenditure of public and private funds for the construction and operation of water purification and sewage treatment works.

The Engineering Division of the Pennsylvania Department of Health is charged, among other things, with the examination of plans of sewerage and sewage treatment works, submitted with applications for issuance of permits; also with making field investigations and recommending to the Commissioner of Health the conditions under which permits may be issued.

Certain fundamental policies have been established to obtain as nearly as possible uniform practice in this work, but it must be borne in mind that the wide range of uses and conditions of the waters of the State makes it

* Harrisburg, Pa.

utterly impracticable to follow detailed policies that will be State-wide in application.

In this discussion, the word, "stream", is considered as including all lakes, ponds, or other waters of the State, and where policies are given for streams used as sources of public water supplies, they apply equally to streams used for other purposes requiring a hygienic standard, such as bathing and ice harvesting.

Classification of Streams.—The problem of administering that part of the "Purity of Waters Act" relative to sewerage has three phases: (1) prevention or abatement of nuisance; (2) protection of sources of public water supply; and (3) maintenance of clean streams.

Even though they are not used as sources of public water supply, streams as a matter of decency should be maintained in a reasonably clean condition, that is, free from sludge deposits on the bed or banks, ebullition of offensive gases, undue turbidity, discoloration, floating solids, or any other nuisance to sight or smell of sewage origin.

In addition to being reasonably clean, streams used as sources of public water supply should provide a raw water sufficiently low in organic and pathogenic bacterial content that it can be purified safely and economically for domestic purposes.

Small streams flowing through highly developed suburban territories or land devoted to park purposes for the pleasure and recreation of the public, demand a high degree of protection and should be so clean that they show no appreciable evidence of sewage.

Degree of Sewage Treatment Required.—The ground-water drained or pumped from coal mines contains considerable sulphuric acid which acts as a germicide on sewage discharged into streams receiving such mine drainage, thus minimizing the danger of creation of nuisance or menace to the public health; also, these streams frequently carry large quantities of fine coal which mask any visible evidence of sewage matters. Therefore, at points reasonably remote from water-works intakes, sewage generally may be discharged untreated into these streams, but preferably from the outlets of the several drainage areas and always into the main current of the stream, in order to prevent local nuisance at the immediate point of discharge and to distribute the sewage more effectively in the stream and thus take full advantage of dilution by the water of the stream and the germicidal action of the acidity.

If the rate of flow and velocity of streams not used as sources of public water supplies are insufficient to assimilate crude sewage inoffensively, then such a degree of treatment should be required as will lighten the load of organic matter on the stream so that the desired conditions of cleanliness will be maintained, even during times of drought.

The assimilating power of the stream should be distributed equitably among the various municipalities along its course, so that one town will not be required to treat its sewage to a high degree because a neighboring community up stream has overtaxed the stream with insufficiently treated sewage.

In the case of streams used as sources of public water supplies, sewage treatment should be required to a sufficient degree to produce an effluent which, after being carried by the stream to the water-works intake below, will not prejudice the safe and economical purification of the water for domestic purposes.

Ample diluting water and sufficient distance between the water-works intake and the place of sewage discharge may reduce the required treatment of sewage to a negligible degree.

Cleaning Streams.—The cleaning of sewage polluted streams not used as sources of public water supplies or the maintaining of such streams in a clean condition should begin at the upper end and progress down stream.

In general, action taken to protect streams used as sources of public water supplies, subject to local conditions, should begin at the first source of sewage contamination above the water-works intake and progress up stream.

Sewer System.—Every municipality should prepare and submit to the State Department of Health for approval, a comprehensive plan of sewerage before undertaking the design of new sewers or the extension of existing sewers, as may be needed from time to time.

In sewering new drainage areas, comparative estimates of cost should be made of:

1.—Collecting sewage and rain water in separate conduits; and

2.—The use of the combined system with interception of all dry-weather flow of sewage and such a percentage of storm water contaminated with sewage as will afford the required degree of protection to the receiving body of water.

In extending existing combined sewers in municipalities having well improved highways, the relative economy of continuing the combined sewers, or of replacing them with separate sewers, should receive careful consideration in connection with the relative protection afforded to the waters of the State.

Contiguous or adjacent municipalities should be urged to give careful study to the possible advantage of entering into agreements for the joint construction and operation of intercepting sewers and sewage treatment works.

Sewage Treatment Works.—In cases where sewage treatment is not required at present, general outline plans of the project should be submitted to the State Department of Health for approval. They need only be prepared in sufficient detail to demonstrate the feasibility of constructing the intercepting sewers and sewage treatment works and to show that the site of the proposed work is properly located, ample in area, and of suitable topography for the erection of treatment works sufficient for the needs.

The advance in the knowledge of sewage treatment in past years indicates the possibility of even greater improvements in the future, and, therefore, it is inadvisable to prepare detail plans until it becomes necessary to construct the works.

If the approved site is likely to be acquired for private use, the municipality should forthwith acquire it, so as to be prepared to treat the sewage when necessary.

In cases where the use and condition of the stream is such that sewage treatment is required, the plans should show in detail that part of the works required for the needs of the present and reasonable future and, in general, outline the extensions required for the increased rate of flow and higher degree of treatment that may be needed in the more distant future.

The operation of oxidizing processes of sewage treatment works, the only purpose of which is to prevent the creation of a nuisance in a stream not used as a source of public water supply, may, on request and with the specific approval of the State Department of Health, be suspended during the colder part of the year, when the greater quantity of dissolved ogygen and lessened biological activity in the stream will permit the discharge of unoxidized sewage.

STREAM POLLUTION AND ITS CONTROL

By Earle B. Phelps,* Affiliate, Am. Soc. C. E.

A complete statement of the problem of the control of stream pollution, aside from its purely legislative and administrative aspects, involves three essential terms capable of expression in common units. These are: The capacity of the stream; existing or contemplated pollution load; and, means of reducing the pollution load.

Every stream has a certain maximum capacity for the biological disposal of common organic wastes. According as local circumstances permit a greater or less depletion of this natural reserve of capacity, with consequent temporary reduction in stream quality, any stream may be said to have an effective working capacity for waste disposal which is a function of the fixed maximum capacity and of the permissible quality depletion. The latter varies according to circumstances and usage, but the lower the standard of minimum acceptable quality the greater the working capacity of the stream for waste disposal.

This working capacity is not a mere capacity for dilution. It comprises all those bio-chemical reactions which make for the self-purification at the stream. A pollution load of stated amount brings about, within a certain distance (the critical point, measured down stream in hours), a definite reduction in stream quality, after which the operation of self-purification becomes predominant and stream quality rises again toward normal. A greater load produces a proportionately greater reduction in quality at the critical point, and subsequent recovery is proportionately delayed.

The maximum load previously referred to, may be defined as that load which, at the critical point, produces conditions inimical to those agencies which bring about self-purification, so that improvement of the stream beyond that point is either greatly delayed or altogether lacking.

Although the available data in this field of investigation are too meager to justify any definite generalization for all streams, yet the methods of investigation have been developed sufficiently at the present time to permit the determination of the fundamental stream constants, the critical point, the maximum capacity, and the rate of self-purification in the case of any stream of known physical characteristics and pollution load. By the aid of these constants the effect of added or diminished pollution load can be determined and, conversely, for any fixed lower limit of quality the working capacity of the stream becomes known. This information is the first prerequisite in any comprehensive program for the control of stream pollution.

A more detailed account of the various subdivisions of this phase of the problem is not possible in this discussion, further than to indicate that it is not simple, but is essentially dependent on sedimentation, additional pollution or dilution in the lower stretches, and re-aeration; and that the latter, the most significant factor of all, is a function of depth, velocity, turbulence, and other physical conditions.

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* Ridgewood, N. J.

It must further be pointed out that all references to stream quality must be in terms of a unit which may also be used in measuring the pollution load and the effect of remedial measures, the so-called oxygen unit, to which reference will be made later.

The second term in the stream-pollution formula is the pollution load. This phase also is complicated in practice and may be dealt with only in general terms in the present summary. Obviously, the oxidation of organic matter in the stream is a time function so that a first approximation to a statement of pollution load must be in terms of total oxidizability and rate of oxidation at each degree of completion. Given this complete oxidation curve, at any stated load, the effect on a stream of known capacity could be determined. Fortunately, the form of this curve has been established so that two determinations on the oxidizability of a waste, taken, say, at 24 and at 48 hours, furnish the necessary and sufficient data. This treatment assumes, however, that all the pollution travels uniformly with the stream up to the point of complete disposal. If a part of the waste is capable of settling and the stream characteristics permit of sedimentation, the oxidation characteristics of the settleable and the non-settleable parts must be dealt with separately with appropriate time factors.

The methods by which the pollution load is measured in the laboratory have served to indicate the only satisfactory unit of measurement for stream quality. The earlier work in this field attempted to deal with pollution in terms of nitrogen and to establish some empirical relation between the content of unoxidized nitrogen in the stream and that elusive factor, stream quality, for which there was no adequate direct measure. These attempts were theoretically unsound and led only to greater difficulties and to such fallacies as the establishment of a constant dilution ratio for sewage in a stream, which was supposed to mark the boundary between safe and unsafe practice. The newer chemistry deals wholly with oxygen units, as these measure pollution load as well as stream quality and are interchangeable between the two when once the fundamental formulas have been derived. The characteristics of the most important polluting agent, domestic sewage, have been quite satisfactorily determined and are referable to the population base, thus eliminating the former difficulty of determining the strength and total volume of the sewage, neither of which *per se* has any but incidental relation to the question of pollution load. Much information has also been accumulated concerning the characteristics of certain of the more important industrial wastes, such as the waste from tanneries, soap works, and others. Here, too, the attempt has been made to refer the pollution load to the reference base, manufactured product. It is obvious that there must be some fairly constant amount of pollution resulting from the cleansing and tanning of a single hide by a standard procedure, regardless of the quantity of water in which the waste is carried to the stream.

With the determination of the stream constants and of the present and prospective pollution loads, the condition of the stream resulting from any assumed increase or decrease in the load may be stated with reasonable approximation. If this condition requires improvement, the degree of

improvement decided on is also capable of expression in terms of the pollution load. The problem then becomes one of treatment of the polluting wastes to the required degree. Except in certain fields, such as the treatment of domestic sewage, this leads to experimental studies of remedial measures which are essentially chemical or biological in nature, rather than engineering. Domestic sewage has been sufficiently investigated so that the engineer now has at his disposal the necessary bio-chemical data to enable him to design works for any specific degree of reduction of the pollution load. A proper appreciation of the basic principles of stream pollution, however, will enable him to design scientifically for the utilization as well as for the protection of the stream, a matter which has been too often overlooked. A considerable field of activity in the remedial treatment of industrial wastes is still open to investigation. Many of the wastes do not lend themselves to the line of procedure suggested by the speaker. The effects of some wastes, such as saw-mill refuse, are purely physical, while the effects of others are purely chemical; the acid residues from certain iron works are an example of the latter. In the majority of cases, however, the effect is bio-chemical, and the results of experimental studies are best stated in terms of oxygen demand pollution units which can be compared directly with the stream capacity similarly stated.

Thus, a scientific basis is laid for the study of this problem of stream conservation which, properly understood, means the maximum utilization of stream resources. In addition to the three scientific aspects of the problem, there are the legislative and administrative aspects without mention of which this discussion would be incomplete. The matter has been placed last, however, because it should rest wholly on a correct understanding of the fundamentals. Much harm has been done to real progress by inverting this natural order and through misdirected legislation, seeking an impossible perfection which has only invited disrespect and lack of enforcement. The successful stream pollution legislation of the future will give wide discretionary power to capable administrative bodies whose duty it will be to determine stream resources and capacities and to distribute these so that a maximum utilization will be made possible. The interstate character of most of the great streams of the country, and the magnitude of the problem in both its investigative and its administrative phases suggests Federal control as a probable final outcome. This is emphasized more particularly by the present failure of most of the States to deal with the problem on any comprehensive basis.

The matter of stream pollution and its control is now essentially one for the health authorities. Historically, it has had conspicuous public health aspects, but time has been lost and attention misdirected by too close adherence to this traditional treatment. The problem of stream pollution is clearly one of conservation of a natural resource with, at times, a public health aspect, which is always paramount when present, and, at other times, an overshadowing commercial and industrial aspect. In short, it is a problem in the maximum utilization of an economic resource and its future development should be placed in the hands of an authority having the widest possible scope and the broadest jurisdiction.

DEPOSITION OF SLUDGES RESULTING FROM SEWAGE DISPOSAL PLANTS.

By T. Chalkley Hatton,* M. Am. Soc. C. E.

The problem of sewage disposal will be presented herein from a different angle than that usually discussed in papers on this subject, and the speaker would like to emphasize a certain viewpoint.

Sewage disposal embraces two distinct problems, the partial purification of the liquor and the final disposition of the resultant sludge. Up to the present time the great bulk of scientific labor has been devoted to solving the first problem, the solution of the latter having been left largely to chance.

The speaker believes that the reason for this is principally economic; that the nuisance to our neighbor using a stream polluted by our sewage demands its partial purification, whereas the disposition of the sludge is more likely to be a nuisance from which we alone may suffer.

Science has discovered several processes of purifying the liquor to any standard which may be required to prevent a nuisance, but the speaker is not so sure that it has been as successful in disposing of the sludge, and he is firmly convinced after many years of intensive study of this phase that sanitary engineers should give it far more consideration than they have.

Engineers cannot expect to make sewage disposal popular with the public if about the disposal plants are spread foul smelling, dirty looking, putrefactive piles of material that breeds flies and vermin which the surrounding population believe infect their properties.

A casual inspection of the sewage disposal plants of England when the sludge is being disposed of on property adjacent to the works convinces the engineer that the work is being only half done, and from personal inspection in 1907, and again in the early part of 1921, the speaker can state positively that, as far as he was able to judge, no improvements worthy of note had been made in the 14 years, and that the nuisance had so multiplied as to threaten the welfare and health of large numbers of people.

Take the experience on this Continent and consider the nuisance being maintained from sludge disposal at Toronto, Ont., Canada, and at Baltimore, Md., two of the largest cities on the Continent which are operating modern sewage disposal plants.

In both these cities, suits have been brought and damages recovered for nuisances arising from sludge disposal. Mr. Marks, of the Pennsylvania State Board of Health, stated to the speaker recently that the present methods of disposing of sludge in the sewage disposal plants in Pennsylvania were becoming an unmitigated nuisance and some other methods would have to be devised.

There is not a sewage sludge produced from any sewage disposal process in which lime is not used, but that has some value as a fertilizer. Such sludge will be sought by the agriculturist if it can be delivered to him in a condition in which he can handle it conveniently, and the speaker believes that is the

* Milwaukee, Wis.

final solution of the problem. It will cost far more in some, in fact, in many instances, to procure this condition than the returns from the sale of the sludge; in fact, there may be instances when no monetary returns whatever will be secured.

Whether it does or does not return a monetary value directly, it does get rid of the filth of the human body in the only logical way by returning it to the soil from whence it came, and indirectly will pay just as the purification of water supplies has paid.

Engineers have had the direct economic viewpoint too much in mind to study this sludge problem properly, and as municipal engineers charged with improving the sanitation and welfare of the communities they must look on it with a broader vision, or sewage disposal as a science will never attain the place it deserves.

The German engineer has had a different viewpoint from the English and American engineer. He has not been as anxious to produce a stable effluent as he has to get rid of the sludge in a way which may add to the wealth of his community, and one who goes about the German sewage disposal plants to-day will be struck with the entire absence of piles of sewage sludges. If one wants to find them he must travel about the neighboring farms where they will be found placed in compost beds to be used to fertilize the soil as the season demands.

The speaker desires to give just one illustration of what can be done by a little useful and intelligent propaganda among the farming community adjacent to a sewage disposal plant.

The city authorities of Rochester have built and operated one of the largest Imhoff tank systems of sewage disposal in the United States. It was designed to dry the sludge produced on beds and dump it into a beautiful valley adjacent to one of the fine parks belonging to the city.

The engineer in charge of the works soon realized that this would finally result in a nuisance, and he went among the fruit growers near-by and induced them to try some of this partly dried sludge on the ground about the trees.

The next season these growers hauled away all the sludge they could and paid a small sum therefor. The demand became so great the following season that the price was increased about 50%, but this increase had no effect and to-day the sludge is being disposed of at about the same price which it costs to de-water and deliver it to the customer. It is believed that in another season or two the authorities will be able to dispose of all the sludge the plant will produce. What this engineer has done, other engineers can do, if they go about it in the right manner, but they must first get the vision that the sludge must be disposed of finally by putting it back on the ground regardless of the cost.

Sludges have been successfully de-watered in several places in America and in Germany. At Providence, R. I., and at Worcester, Mass., filter presses have been used for many years, while in Germany centrifuges have been used with equal success. In Baltimore, during the past five years, much of the sludge has been de-watered at 12 and 15% and sold as a fertilizer, and at present about 25 tons per day of dry material is thus being produced.

Whether or not the Baltimore experiment is self-supporting, the speaker cannot state, as he knows nothing of the cost of the overheads. The original contractor who undertook this work 4 or 5 years ago is still on the job, although his original contract was for two years only. How he continues with the present lack of demand for fertilizer is more than the speaker can tell, but he is continuing nevertheless. The contract thus far has been profitable to the city.

In Milwaukee this sludge-disposal problem has been studied for five years, and although it has not been solved satisfactorily, the point has been reached where the moisture content has been reduced by sludge presses and dryers to 10%, and if the demand for fertilizer is restored to pre-war conditions it will be possible to dispose of the sludge for at least what it costs to produce it and probably a profit will be enjoyed.

THE DILUTION FACTOR*.

By Langdon Pearse,† M. Am. Soc. C. E.

Discussion of stream pollution in the early days appears to have centered mainly on the standpoint of nuisance. As concentration of population has increased, together with the load on the stream (or other body of water), other problems have entered, such as the effect of the sewage and sludge on fish life, shellfish, and bathing. In more recent years, in many localities, consideration has been demanded for the degree of loading which can be handled by a water purification plant. From this gradual development, two widely distinct dilution factors may be said to have arisen, one relating to prevention of nuisance, the other due to prevention of undue load on the water purification plant. On the first of these factors considerable data are available; on the second comparatively little is known as yet. The purpose of this brief discussion is not so much to summarize existing data, as to point out the need for well considered investigations over considerable periods of time on stream pollution problems, and to cover quantitatively the subject of loads on water purification plants.

In the problem of stream pollution, the dilution factor of nuisance is of prime importance and has provoked discussion covering the past 35 years. Many authorities have endeavored to state this factor on a basis of cubic feet per second per 1 000 population. These bases have been largely varied by the type of city studied, and the character of the stream, as shown by results in Massachusetts and elsewhere. In general, a range of from 4 to 7 cu. ft. per sec. per 1 000 population has been indicated, with a lower limit of 2 and an upper limit of 10 cu. ft., according to the authority.

In England, high dilutions have been recommended recently by the Royal Commission, as noted by Mr. A. J. Martin, upward of 27.8 cu. ft. per sec. per 1 000 population having been mentioned.

In the problem of the Sanitary District of Chicago, an expression was early sought for a somewhat different angle of attack, namely, for the amount of oxygen required. For this determination the bio-chemical oxygen demand test has seemed to be most favorable. Continued tests on a large sewer serving 300 000 people, with no marked industrial wastes, have given an equivalent of 0.22 lb. of oxygen per capita for complete oxidation. This requirement is based on an incubation period of 10 days, in sealed bottles, and takes no account of re-aeration factors.

Inasmuch as the oxygen content of water varies with the temperature, for translation into more familiar equivalents, Table 1, based on the 0.22 lb. per capita may be helpful.

These figures are indicative of what would be needed, were no re-aeration to occur. The re-aeration factor should reduce the flows required per 1 000 population.

Of the oxygen requirement roughly about from 20 to 30% seems to be necessary in the first 24 hours.

* Paper read by Dr. F. W. Mohlman.

† Chicago, Ill.

TABLE 1.

Dissolved oxygen, in parts per million.	Temperature corresponding to dissolved oxygen saturation, in degrees Fahrenheit.	Flow required in cubic feet per second per 1 000 population.
6	108	6.80
8	81	5.09
10	60	4.07
12	46	3.40
14	35	2.92

Present observations on the Main Channel at Chicago appear to indicate that a higher dilution than the minimum of 3.33 cu. ft. per sec. per 1 000 population is required to dilute the raw sewage of domestic origin, in the summer; between 4 and 5 cu. ft. per sec. now seems to be nearer the requirement. With an industrial load at times more than 50% of the human load, the need of much higher dilution for both domestic and industrial sewage combined is apparent. The exact figure will depend on re-aeration factors and seasonal conditions, such as temperature and the extent of the ice sheet, as well as the standard of dissolved oxygen to be maintained at critical points.

Data on re-aeration are needed. The extent will depend on the swiftness and turbulence of the stream. In traveling from Marseilles to Chillicothe, Ill., a distance of 65 miles, the Illinois River absorbed about 205 000 lb. of oxygen from the atmosphere, on one test, approximately 3.8 parts per million.

From the standpoint of sewage dilution, the problem is reduced to the determination of the amount of pollution to be handled and the amount of oxygen available plus re-aeration. To make a balance, additional dilution, or supplementary treatment, may be required. The exact solution will hinge also on the rate of oxidation and the amount of residual oxygen desired at critical points.

The amount of residual oxygen to be preserved will depend on the conditions to be met. Where nuisance alone is to be avoided, there seems to be little doubt but that some residual oxygen must be present. This may not prove to be sufficient to prevent odors from decomposing sludge banks.

From the standpoint of fish life, considerable oxygen is required. While fish may live in low oxygen content for a short time, they cannot survive long. Fish specialists of to-day suggest not only the study of the oxygen content, but also the carbon dioxide and the hydrogen-ion concentration.

A number of years ago, Dr. Arthur Lederer and the writer made an extended investigation of the fish question for the Sanitary District of Chicago, with the following conclusions:

1.—Fish life is affected by material in suspension and solution, and the variations thereof.

2.—Material in suspension may cause disease by fungus growths or death by mechanical stoppage of the gills or suffocation thereby. The character, size, and quantity of the material must be considered. A careless and continuous handling of fish may induce fungus growth through the abrasion of the slimy, protective coating of the body, thus allowing foreign bodies to adhere.

3.—Material in solution includes mineral and organic matter and gases. The investigation covered only dissolved oxygen, which, of the gases in solution, was the most important in this case.

4.—For continuous fish life, a content of dissolved oxygen of at least 2 to 3 parts per million is required by practically all the fish used in the investigation.

5.—The fish studied may live for very short periods in a content of dissolved oxygen of between 1 and 2 parts per million. This provides time for the fish to escape to better conditions, if any are available.

6.—English observations, as well as German experiments, indicate that, in general, 25 parts per million of dissolved oxygen are about the minimum for fish life and that for continuous thriving fish life more is desirable. In the light of the experiments in America it appears that many species will live in 25 parts per million, but that more is desirable.

7.—Sudden changes in the character of the environment are undesirable and may cause the death of the weaker specimens.

8.—Of the fish experimented on, the vitality or their relative ability to withstand low contents of dissolved oxygen, are roughly in the order designated as follows, the strongest being listed first, the weakest last: German carp (from Illinois River); catfish and bullhead (from Illinois River); black bass (from Illinois River); yellow perch (from Lake Michigan); and sunfish (from Lake Michigan). The pickerel and golden shiner apparently are to be classed with the sunfish. Further data, however, are desired, as the tests on these two species are not as extended.

At the present time an extended joint investigation by the U. S. Public Health Service and the Sanitary District of Chicago is under way, which will cover in detail the self-purification of the sewage of the District from Lake Michigan to the Mississippi. Many of the points suggested herein will then be elucidated. The effect of re-aeration, temperature, ice sheet, time or length of travel, and sludge travel, are to be studied, and existing data will be summarized. With the work already done by the Public Health Service on the Potomac River and the Ohio River, this third investigation should provide useful data for all students of stream pollution problems.

The dilution required to prevent an undue load on water purification plants appears to have been first cast in definite form in the work of the International Joint Commission. The standard suggested was that the average load on such a plant should be such that the raw water would not contain as a yearly average more than 500 *B. coli* per 100 cu. cm., or, as translated by Professor Phelps, a dilution of 4 cu. ft. per sec. per capita or greater should not produce an undue load.

In a paper entitled "The Loading of Filter Plants", H. W. Streeter, Assoc. M. Am. Soc. C. E., has discussed the problem in some detail from the standpoint of the efficiency of the filter plant, corroborating in general the report of the International Joint Commission. However, he suggests the need of more data and a definite adoption of a standard for filtered water.

Many filter plants delivering a reasonably safe potable water, judged by the typhoid fever death rate, are handling raw waters more polluted than those mentioned. How safe they will be, experience alone will tell. The need at present is for careful standardized tests of such plants in order to determine the data along common lines. The U. S. Public Health Service has made an extended survey of the Ohio River Basin, and more studies might be made

available from scattered plants, by the more general adoptions of standard methods of analysis and report.

Such data will be helpful in solving the problems ahead, not only on the Great Lakes, but on many of the large rivers of the United States. Further, such data will be helpful in crystallizing the policy of various State board of health authorities who have jurisdiction over different parts of the same stream, as in the case of the Missouri River from Kansas City to St. Louis, Mo.

There is, at the present time, need of a sane, well-considered viewpoint, looking ahead, not only from the standpoint of nuisance, but from the standpoint of fish life, and in many cases of drinking water. Various organizations of sportsmen, nature lovers, conservationists, and others are urging the restoration of streams to their former condition of virgin purity. How far it is necessary or possible to go in each individual case is a question for the engineer to solve. For the solution, every available experience helps.

The writer desires to acknowledge the courtesy of the Trustees of the Sanitary District of Chicago and of Albert W. Dilling, Assoc. M. Am. Soc. C. E., Chief Engineer, for the use of certain data contained herein.

PREVENTION OF MISUSE OF SEWERS

By W. H. Dittoe,* M. Am. Soc. C. E.

The purpose of this discussion is to call attention to the abuse of sewerage and sewage disposal systems resulting from failure of municipal officials to control properly the establishment and use of connections to sewers. Sanitary engineers generally have deplored this condition of affairs and, as a class, are in agreement that better control should be provided. However, they have been prone to leave entirely to municipal officials the solution of this problem and with few exceptions have failed to recognize that it is essentially their duty to take the lead in establishing proper control. This discussion will attempt to show that sanitary engineers must not only recognize the importance of preventing abuse of sewerage and sewage disposal systems, but must undertake the problem of prevention if the expected efficiency of such improvements is to be realized. The speaker will not recite instances to demonstrate the evil effects of misuse of sewerage systems, with which all engineers are familiar, but will confine this discussion to general statements with the hope of arousing an interest on the part of engineers in attacking a solution of this problem.

One of the most important factors reducing the efficiency and value of systems of sewerage and sewage disposal is the misuse of sewers. Sufficient emphasis has not been placed on this subject, and rarely is it found that a municipality enforces a strict policy regarding the use of sewers. Considerable effort is expended by engineers in designing sewerage systems and sewage disposal works and, as a basis for such design, the volume and character of the sewage flow must be known or estimated. Therefore, if these factors are disturbed appreciably, the improvements will not be used under the conditions for which they were designed. The result will be a shorter life of the system as a whole, impairment of its efficiency, and generally unsatisfactory results. The misuse of sewers is also an important fault affecting the successful operation of sewage disposal works, and therefore the problem of prevention of stream pollution. It is useless to design sewerage improvements on an assumption that certain maximum rates of flow will occur and that the sewage will be of a certain character, unless the construction of such works is followed by the enforcement of a definite policy which will insure against exceeding such rates or changing such character.

Sewers are designed for definite purposes and when used for other purposes may be said to be misused. Storm drains are misused if they receive sewage, industrial wastes, or other waste of objectionable character. Combined sewers are properly used for the removal of practically all classes of liquid wastes, but are misused if they receive industrial wastes affecting the sewerage system or process of sewage treatment. Sanitary sewers, as the name implies, are for sanitary purposes only, and are misused if they receive drainage from the surface and roofs, subsoil drainage such as may be admitted by building foundation drains and through open or leaky joints, and industrial wastes of a character to affect the sewerage system or treatment process.

* Columbus, Ohio.

The effect of the misuse of sewers is frequently quite serious. Sewage discharged into storm sewers causes nuisances at outlets and offensive odors through street inlets. If such practice is permitted, the benefits to be expected from a separate sewerage system are not realized. The admission of surface and subsoil drainage to sanitary sewers overtaxes the sewerage system, resulting in cellar flooding and damage to sewers, and overburdens the pumping equipment and sewage treatment works, necessitating by-passing of the flow, impairing the efficiency of the plant, and frequently causing rapid deterioration. Industrial wastes often cause clogging or may actually destroy the sewers if the wastes have solvent properties. Many industrial wastes also interfere seriously with the efficiency of sewage treatment plants. Of the more troublesome industrial wastes, may be mentioned: Wastes from tanneries and glue factories containing hair and lime; wastes from textile industries containing cloth, fibrous material, and objectionable compounds in solution; wastes from canning factories containing vegetable particles; wastes from stockyards containing manure; wastes from packing plants containing animal offal; gasoline wastes from garages; acid wastes from metal industries; wastes from milk industries; and gas-house wastes. Such wastes should be treated properly prior to their discharge into the public sewers, or they should be excluded entirely.

It seems apparent that the evils resulting from the misuse of sewers are of sufficient importance to warrant an effort to prevent it. It has been accepted by many engineers that the misuse particularly of sanitary sewers to carry storm water, is inevitable and cannot be prevented, and this conclusion has produced a strong argument for the selection of the combined system. It is true that the combined system can rarely be misused and that from this standpoint it is preferable; however, it does not appear sound to conclude that it is impracticable or impossible to secure the proper use of separate systems. In fact, separate systems now in existence will continue in use and new systems will be built; therefore, engineers and city officials cannot avoid the responsibility for securing their proper use. The industrial waste menace is present and must be controlled regardless of the sewer system used.

It is obvious that the proper use of sewers cannot be secured without strict enforcement of ordinances and regulations by the proper municipal officials. The sewer contractor and the property owner cannot be expected to realize the importance of using the sewers properly and for the purposes for which they were designed to function, and, therefore, they must be controlled in order that the public may not suffer from their mistakes. Ordinarily, munipical officials themselves do not appreciate the necessity of protecting sewers and sewage treatment plants from abuse, and, therefore, are not in a position to initiate suitable regulations. It seems apparent that it becomes the duty of sanitary engineers to dictate such regulations and to see that they are adopted. This function of the engineer is as important as the design and supervision of construction of the improvement, and if it is not performed it may truly be said that the work of the engineer has not been complete. Ordinarily, the engineer who has designed and supervised the construction of a sewerage system and a sewage treatment plant furnishes definite instruc-

tions in regard to the operation and maintenance of the system, and he should likewise furnish a definite program for preventing misuse which may defeat the purpose of the improvement.

Many municipalities have satisfactory ordinances and regulations, but fail to enforce them. Such ordinances usually require permits for connections to the sewers and provide for inspection of the connection by a representative of the municipality after the contractor has completed the construction work and before the trench is filled. Theoretically, this control should be sufficient, but too frequently the results are far from satisfactory. In many instances the construction work is faulty, joints are made imperfectly, admitting ground-water to the sewer, the inspection is neglected or performed carelessly, improper wastes are admitted, and no proper record of the connection is maintained. When this system of control is started improperly, it is difficult to correct it and make it efficient, and usually the conditions become worse rather than better, until the sewer system is generally abused.

It seems necessary that municipalities provide a more immediate and direct control of the use of sewers, if sewerage systems are to be managed and maintained as they should be. The most logical and effective method of accomplishing this is the construction by the municipality of all connections to the public sewers from the building to the street sewer and the continuation of municipal control over such connections after they are constructed. The sewer department would organize its construction gangs for this work or would enter into annual contracts with responsible contractors, and the property owner would pay to the city the cost of construction, inspection, and recording.

When local treatment of industrial wastes is necessary to protect the sewerage system or sewage disposal works, such treatment would be provided by the industry and the effluent received into the sewer system under proper control. The connection could be equipped with an inspection hole to permit subsequent examination by the sewer department of wastes discharged through it, and the discharge of prohibited wastes could thus be detected. It is believed that this method of construction would insure better construction of the connection at lower cost, would largely prevent the misuse of sewers, and would assist in securing efficient operation of sewage treatment processes. Incidentally, it would probably arouse a more lively interest on the part of the city officials in the management and maintenance of the sewerage systems and would likewise remind the public that the system is an important feature of the community development and must be controlled in a business-like manner if its value is to be realized.

DISCUSSION ON STREAM POLLUTION AND SEWAGE DISPOSAL

By Messrs. Harrison P. Eddy, J. Frederick Jackson, F. A. Dallyn, W. F. Wells, Alexander Potter, Edward S. Rankin, and S. John Scacciaferro.

Harrison P. Eddy,* M. Am. Soc. C. E.—One item not mentioned by Mr. Hammond is important, namely, the effect of temperature on the period which should be provided for sludge storage in the digestion tank. Presumably, the eight months mentioned refers to the climatic conditions of Brooklyn, N. Y. In the north, digestion is confined in large measure to the warmer months of the summer, and storage of the sludge is required over a considerable part of the year. The same treatment in a warm climate where digestion can proceed throughout the year at a nearly uniform rate, may require a very different period of sludge storage, and, therefore, it would seem important to consider temperature conditions as well as the quantity of solids in the sewage.

The point presented by Mr. Stevenson in connection with the seasonal operation of sewage treatment plants is also important. Although, from some points of view, it is desirable to deal with this subject in an idealistic manner, it is probably not wise to disregard economic conditions. The following illustration may be cited to show that it is practical to operate a plant according to climatic and physical conditions. A certain plant on a relatively small stream was equipped with various means of treating industrial wastes. These wastes are first screened and then passed through sedimentation tanks, after which they are treated by chemical precipitation; and, finally, the treated liquor is passed through sand filters. On a branch of the stream there is a reservoir with a capacity of about 300 000 000 or 400 000 000 gal. For a number of years this plant has been operated according to climatic conditions. In winter, screening and sedimentation have appeared to be a reasonable treatment; in the latter part of May, as the stream flow is reduced, the sand filters are put into use in order to treat a substantial part of the wastes, and by that improved treatment the river is maintained in a relatively good condition. Still, later, as the flow in the river is reduced further, water is drawn from the reservoir, and parts of the unfiltered wastes are diluted with this flow in proportion to their quantity. During the period of hot weather of August and much reduced flow of the river in September, it is customary to take advantage of chemical treatment, and the wastes are treated with sulphate of alumina. The treated liquor is allowed to settle, and a portion of the effluent is passed through the sand filters, and the remainder is diluted with water from the reservoir and discharged directly into the river. On one occasion, during an extremely dry season when the supply of water became exhausted, sodium nitrate was added to the effluent to supply oxygen to the stream. This is rather an elaborate treatment, but it is far less costly than treating all the wastes throughout the year, in the manner required for the unusual conditions of the summer season.

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* Boston, Mass.

The subject of separate systems of sewerage and drainage, presented by Mr. Dittoe, is of surpassing importance. About thirty years ago a Massachusetts city with a population of approximately 20 000 built a separate system of sewers with extreme care. The pipe joints were made very tight, and the system was apparently an excellent piece of workmanship. In recent times, measurements of the dry-weather flow at the treatment plant have shown less than 300 000 gal. in 24 hours. In time of storm, the flow has risen nearly to 3 000 000 gal. in 24 hours. A thorough investigation to ascertain whether it was possible to correct that condition, has shown that, except in the case of one or two sewers which could be and have been improved, it is practically impossible to better the situation.

In a certain part of another city of relatively large size, separate sewers and drains were built by a commission independent of the City Council. On their completion, they were turned over to the Board of Works with an explanation, that roof drains were not to be connected with the sewers, and that sewage was not to be discharged into the storm drains. Within a short time sewage was discharged into the drains, and water from the roofs was discharged into the separate sewers. It has been practically impossible to prevent these conditions, notwithstanding the fact that the city has a City Engineer who undoubtedly knows the needs of the case, but is impotent to remedy the conditions.

A number of similar cases might be mentioned, where separate systems of sewers have been so overburdened with the discharge from roofs and even from streets, where the street-water connections have been made by the city, that the systems have become partial, if not complete, failures. The problem is so serious that it is questionable in many cases whether separate systems of sewers should be constructed. Undoubtedly, separate systems will be constructed in the future, and it is to be hoped that means will be devised for using the sewers and the drains only for the purposes for which they were built.

About thirty years ago, separate sewers were built in a certain part of Worcester, Mass., and the care and diligence required to assure their proper use, is a serious task. It may be of interest to mention the routine by which it has been accomplished. The connections between the sewers in the buildings are built by drain layers who are licensed by the Board of Aldermen, on the written approval of the engineer in charge of the sewerage system, and occasions have arisen when that approval has been withheld. Connections are made under the direction of the engineer in charge, who inspects the work. The plumbers are licensed by the local Board of Health and, before they make any plumbing changes or construct new plumbing, are required to file a drawing showing what they propose to do. That drawing is filed with the local Board of Health which turns it over to the Sewer Department the approval of which is required before a permit to do the work is issued. After the plumber has completed the work, it is inspected by the Department of Health. That, however, was not sufficient, because where both sewers and

drains are required and the pipe lines are carried to the inside of the cellar wall by the drain layer, the plumber has no means of ascertaining which is the sewer and which is the drain. Finally, it was decided that the hub of the drain pipe should be painted white and that of the sewer pipe remain its natural black color. Even this complicated control did not completely eliminate wrong connections, made either by mistake or surreptitiously; and thus far it would seem practically impossible to prevent all such connections. House owners, without leave or license, will connect roof waters with sanitary pipes, and it is almost impossible to find out where such connections have been made. Mr. Dittoe has certainly presented a very serious question for consideration.

For many years, it has been the practice at Worcester, to construct the separate system in the outlying districts, and portions of the central part of the city have been resewered according to the separate plan. The speaker knows of no changes in the combined system since 1900, and he does not know of any further plans for changes to the separate system.

J. Frederick Jackson,* M. Am. Soc. C. E.—The speaker assumes that the object of this Symposium is to hasten that much sought day when pollution will be removed from the streams in the United States. It has been stated that methods of sewage and sludge disposal should be adapted to the different classes of sewage being treated. As a general statement this is true, but engineers have not by any means been successful in demonstrating this fact to the public at large, and it is still possible for any one with some new method, to approach city officials having the final say in such matters and with a plausible line of talk apparently convince them that they have the one panacea which will solve all sewage disposal problems.

The Industrial Wastes Board of Connecticut which was recently legislated out of office, was, strange to say, severely criticized, because it would not endorse a bill the provisions of which seemed so drastic as to make enforcement of them impossible.

One of the sensational papers in Connecticut recently carried a full-page article in which it was stated that a problem in stream pollution which the Industrial Wastes Board had had under investigation for three years and had failed to find a solution, had been solved in one week by a patented process controlled by a party outside the State. The reporter who prepared the article stated that while inspecting the plant, the patentee who was demonstrating the operation of the process, took a bottle of the wastes, added some ingredients, shook it up and down two or three times, and then holding it up to his view asked, "Are you satisfied?" He replied, "I am." Such ocular demonstrations of the solution of troubles of this kind apparently are effective with certain people.

Personally, the speaker believes that no successful solution of the problem of stream pollution will ever be accomplished in the United States, or elsewhere, until a strong public support is carefully and conscientiously built up beforehand. There have been a number of remarkably careful and reliable investiga-

* New Haven, Conn.

tions of stream pollution. What the public wants is a demonstration of some particular problem that has been solved or of some particular stream, cleaned up. Investigations, no matter how thorough, apparently have little effect on legislatures. At the hearings before the Judiciary Committee in Connecticut, the Industrial Wastes Board was told, "investigations of stream pollution have been going on in this State for thirty years, your particular Committee has been investigating the problem for four years, and you have spent \$75 000. What have you accomplished? Nothing. We think that there have been investigations enough. What we want is the removal of the pollution from the streams." The bill which was under discussion provided that all pollution of the streams of Connecticut should cease by 1923, and this bill would have been passed by the Legislature except that, as one of the Judiciary Committee stated recently, the members of the Committee could not frame it so that it would be operative. If members of the Judiciary Committee cannot frame a bill to meet a proposition of this kind, how is it expected that engineers are going to do so? Because the engineers of the Industrial Wastes Board refused to set any time limit as to when they thought the pollution of the streams of the State should cease, the members of the Committee said, "Then you have failed."

As a result of experience in Connecticut, the speaker would call attention to the inadvisability of delaying too long, the issuing of reports. The report, in most of the important investigations in this country on stream pollution, is cold by the time it is published, and it seems that if some method could be provided whereby reports or portions of them could be issued while the problem is alive and before the public, it would be of great assistance in effecting a solution.

To discuss in detail many of the excellent points which have been presented would take up too much time, but the speaker did wish to give the members an idea of some of the difficulties outside of stream pollution problems in themselves, which engineers in Connecticut have encountered in their investigations.

F. A. DALLYN,* ESQ.—One of the most encouraging signs in this discussion is the emphasis placed on the fact that sanitary engineers are now including biologists in their forces. The recognition of the part that biology is to play in the future of stream protection, in the operation of disposal plants, and in the degree of purification required, is the greatest step that has been taken by engineers in a long time, and it is hoped that a great deal of the time in future meetings of engineering societies will be left open to the researches of biologists. In his experience, the speaker has found no easier way of convincing industries of the facts relating to any situation than by putting them on a scientific basis. Nowadays, practically all large industries employ scientific experts; and when the matter is put before them in convincing terms which they understand, they appreciate the situation better than when engineers spoke in general terms of what they could do under such conditions.

* Provincial San. Engr., Toronto, Ont., Canada.

W. F. WELLS,* ESQ.—As a biologist, the speaker will discuss briefly some phases of stream pollution, which are not necessarily engineering, but which are vitally connected with the problem of improving the streams in conserving the qualities of their waters. Mr. Jackson has brought up the point that although engineers have a great many data on the subject of the purification of wastes, when the public becomes interested in this subject as relating to the purity of water, it is at a loss to find any information which it can understand. In the last year or two, this question has become very important, because of the interest taken by those who are concerned in fish, commercially and for pleasure, as well as many other interests, such as bathing, gunning, etc., which are legitimate uses of these great resources of the people.

Recently, the speaker attended a conference in Washington, D. C., called by Herbert C. Hoover, M. Am. Soc. C. E., Secretary of Commerce, to consider problems brought about by the pollution of the coastal waters as affecting the fisheries. Incidentally, Senator Frelinghuysen, of New Jersey, pointed out the great importance of this subject, dwelling on it at considerable length as relating to the insurance business, namely, the higher rates on water-front property, which are brought about by oil pollution. Much has been heard about oil pollution recently. There are many phases and many angles connected with the problem of conserving the quality of the waters, and the people themselves are vitally interested, as shown whenever any one discusses the subject before those who are not particularly interested in the purification of wastes.

In fish conventions, the subject is given much attention, and, as Mr. Jackson has stated, the chances are that somebody will appear with a panacea for all the ills attendant on pollution; and the interest which these enlightened individuals take in the subject, and the ease with which they are convinced that there should be no such thing as pollution, is rather disheartening to one who is acquainted with the results which have attended such efforts in the past, and which are common knowledge among the Engineering Profession, but never get outside of it. The only way the question can be developed to the importance which it rightfully deserves, is by promoting in some manner public interest which is hungry for action of some sort, by uniting, as far as possible, the interests relating to the pollution of waters, by trying to get together those who are working on it and those who are interested in it, and by devising some means by which the knowledge that engineers have, may be given to the people who are concerned and whose resources are gradually being monopolized by individuals, industries, and particular communities. It is rather difficult to get such co-operation.

The speaker discussed the problem at the conference in Washington along that line, but his opinion did not receive much encouragement among officials, and he doubts whether it would receive the necessary attention among the Engineering Profession. However, the steps which are being taken, such as those mentioned by Professor Phelps, to develop a language of pollution—a technical language, terms in which the quality of waters can be measured and in which the purification of wastes can be stated—that will bring out

* Albany, N. Y.

the problem concretely, so that it can be handled, will tend, in the long run, to bring together those who are studying this subject, those working on studies of fish life, as well as those working along other lines. If they have some medium by which they can get into contact, ultimately the solution of stream pollution will be realized, and a great advance will be made not only with respect to general questions relating to the subject, but in the support that will be given the Engineering Profession in carrying out its essential work.

ALEXANDER POTTER,* ASSOC. M. AM. SOC. C. E.—It is impossible to over-emphasize the importance of securing tight joints in house laterals and of eliminating all waste water not intended to be cared for by a sanitary sewer system.

Engineers too frequently have labored faithfully on the construction of the street mains of a sanitary sewer system only to find their efforts defeated afterward by the careless and indifferent construction of house connections. When these conditions occur, it is almost impossible later to correct them, short of reconstruction of most of the house laterals. The speaker recalls an effort at Summit, N. J., some years ago, to eliminate the leakage in the system, which came very largely from the house connections. Carelessness in house connection construction is not immediately noticeable in a gravity system, but when, as in the case of Summit, it became necessary to change from a gravity system to a pumping system which operated against a 200-ft. head, the elimination of all leakage was essential.

Although the exclusion of house leaders made some reduction in the quantity of waste, the bulk of the leakage came directly from the open joints of the laterals themselves. That much of the infiltration in sanitary sewers comes in through manhole covers is a popular conception, but the two or three days' lag in abnormal flow after a storm indicates a more indirect source.

Efforts made by municipal engineers to provide tight sewers have been thwarted, often purposely by town officials in their endeavor to make improper use of sanitary sewers, that is, for cellar drainage, roof leaders, yard drainage, and here and there a street catch-basin. The protest of the engineer and his pointing out the disastrous conditions which inevitably result from the improper use of house sanitary sewers, has little or no effect in many cases on the officials responsible for the maintenance of the sewer.

In one municipality with which the speaker was identified, the officials deliberately attempted to secure storm sewers by connecting leaders, yard and cellar drains to the sanitary sewer system. Expostulation only brought the retort: "The sooner the sanitary sewer is used to capacity, the sooner we will get an adequate system". This argument might be valid if the house laterals could be transferred from the sanitary sewers to the storm sewers at will.

This disregard of the purposes for which the sewer is built causes overflow at low points and along outlet lines long before the normal capacity of the sewer has been reached. The resultant pollution is tolerated for years and

* New York City.

is often greater in volume than that contributed from the legitimate overflow from combined sewers during heavy rains.

The speaker, knowing that these conditions exist and that they are next to impossible to remedy, has believed for many years that the adoption of the combined sewers with dry-weather interceptors is preferable to the strictly sanitary sewer. In many cases, only sufficient funds are available for sanitary sewers. Then, of course, the adoption of the separate system is logical and proper.

As far as the effect of the final disposal of the sewage is concerned, the disadvantage of the greater fluctuation both in the quantity and character of the sewage reaching the disposal plant and of the occasional discharge of diluted sewage at storm overflows is offset by the advantage of excluding from the watercourse the filth accumulating in the streets during protracted dry spells, which exclusion cannot be affected when sanitary sewers are used.

The speaker is pleased to note a change in the policy of the health departments of many States in giving their approval to the use of the combined system, and it is hoped that this policy will have a wider range.

Edward S. Rankin,* M. Am. Soc. C. E.—As another instance of the difficulties encountered by the engineer in preventing the improper use of sewers, the speaker's experience in Newark, N. J., may be of interest.

All the older sewers of Newark, which comprise about two-thirds of the city system, are built on the combined plan, but, in a number of the outlying sections, the separate system has been used during the last 20 years.

On the separate system, the average citizen cannot understand why he should not be allowed the same privileges as the citizen on the combined system, and be permitted to drain roof water into the sewer. For some years one of the members of the old Board of Works was a plumber who apparently held the same views. An investigation, a few years ago, disclosed a large number of leaders connected with these sanitary sewers, several of them having been made by this City Official.

Under the present commission form of government, this matter is entirely controlled by the Engineering Department, and the practice referred to has been almost eliminated.

S. John Scacciaferro,† Jun. Am. Soc. C. E.—The speaker was much interested in the discussion by Langdon Pearse, M. Am. Soc. C. E. With reference to the oxygen requirements of fish life, the results of some recent experiments performed under the direction of Professor C. L. Walker, Affiliate, Am. Soc. C. E., at Cornell University, for the New York Milk Conference, may be of interest. The experiments in general indicate that the ordinary milk waste is not toxic to fish, and is not detrimental to them except as it utilizes available oxygen; as long as the oxygen content is not reduced below 1 part per million, the fish will continue to live. The fish used were bass, trout, and the ordinary fish found in the lakes of Central New York State. During October, the trout and bass were kept in undiluted milk waste, both

* Newark, N. J.

† Clifton, N. J.

raw and after septic tank treatment, without any evident detrimental effect. The oxygen content, however, was about 4 parts per million.

Regarding the bio-chemical oxygen-demand test, the speaker agrees with Mr. Pearse's statement as to the usefulness and value of the test; he thinks, however, that the modified "excess oxygen" method developed by Messrs. H. B. Hommon and E. J. Theriault, (the method used in Mr. Pearse's experiments), requires futher study, modification, and improvement. This statement is prompted by the results of some experiments made in the Sanitary Laboratory of the College of Civil Engineering, Cornell University in 1920, the procedure described by Hommon* being carefully followed. The wastes used were: Milk-can and bottle washings before and after treatment in septic and Imhoff tanks, the effluents of these tanks after treatment in sand and percolating filters, cheddar cheese whey, and the sewage of Ithaca, N. Y., before and after septic tank treatment. Tests at 1, 2, 5, and 10-day incubations at 20° cent., were made of fifty-two series, each series consisting of three dilutions of a waste (except for the sand filter effluent in which case the undiluted waste was used). Tap water with an average initial oxygen content of 6.6 parts per million was used for dilution.

The results obtained were very discouraging, their outstanding features being a general inconsistency and the presence, at times, of greater amounts of dissolved oxygen on one day than on the day preceding. This last condition occurred frequently, despite the care taken to prevent any possible re-aeration or loss of iodine in the titrations. Due to the rather limited number of tests, and the wide variation in the results, no definite conclusions could be drawn. The results obtained, however, indicated that:

(1).—The "excess oxygen" method for the determination of the bio-chemical oxygen demand, although simple of performance, does not yield consistent results.

(2).—The bio-chemical oxygen demand is not independent of the dilution, as stated by Mr. Theriault,† but varies with the concentration of the waste, being higher for the lower concentrations.

(3).—No prediction of the probable demand at the longer incubation periods can be made from the results at the shorter periods (again failing to corroborate a statement of Mr. Theriault,† that from the results of the 5-day or possibly the 2-day incubation, the demand for the longer incubation periods could be predicted).

Despite the results obtained, the speaker still firmly believes that the test is important, and that further experiments are desirable, which will improve the "excess oxygen" method for the determination of the bio-chemical oxygen demand.

* *Public Health Bulletin,* No. 97.

† Public Health Report for May 7th, 1920.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

WATER SUPPLY AND WATER PURIFICATION*

A SYMPOSIUM

By Messrs. George C. Whipple, Allen Hazen, C. A. Emerson, Jr., C.-E. A. Winslow, C. A. Holmquist, Robert Spurr Weston and Samuel A. Greeley.

With Discussion by Messrs. Louis L. Tribus, G. F. Catlett and John R. Baylis.

* Presented at the meeting of November 17th, 1921.

HISTORY OF WATER PURIFICATION

By George C. Whipple,* M. Am. Soc. C. E.

The history of water purification in the United States is intimately connected with the general movement to improve sanitation and public health, as well as with the rise of modern science. No excuse, therefore, is necessary for referring to these subjects by way of introduction.

The great sanitary awakening occurred in England about the middle of the 19th Century. Following the social uplift, sometimes called the New Humanity, which occurred in the Twenties and Thirties, came the sanitary surveys of Sir Robert Rawlinson, the public health activities of Sir John Simon, the statistical studies of Edwin Chadwick, and, in the Forties, the medical investigations of Dr. Southward Smith. The English public health system dates from 1848. The new public health inspiration crossed the Atlantic and, in the late Forties, occurred the investigations of Lemuel Shattuck, which culminated in that remarkabe document, the Report of the Massachusetts Sanitary Commission† of 1850. The movement, however, became side-tracked in the United States, because more important issues were before the public.

During and after the Civil War the problems of sanitation were attacked anew. A Citizens' Association made an investigation of the sanitary condition of New York City in 1865. The Massachusetts State Board of Health was established in 1869. The American Association of Public Health was founded in 1872. Meantime, a new demand for public health measures had arisen. The Civil War had left an aftermath of disease. In particular, typhoid fever which had been so prevalent in the armies, was scattered throughout the country. Many a returned soldier had become unwittingly a typhoid carrier, and the typhoid fever death rates were very high. In Boston, from 1855 to 1860, the typhoid rates were between 40 and 50 per 100 000; during the Sixties they were above 60 for one-third of the time; in 1872, the rate was 86 per 100 000—high figures for a city like Boston. The rates in other cities were equally high, sometimes much higher.

It should be remembered, also, that it was during the last half of the 19th Century that modern science began its remarkable development. Several events marked the new era, the most important perhaps being the publication of Darwin's "Origin of Species" in 1859. With it, came the wide application of the inductive system of reasoning, of the experimental method. All sciences went forward as a result of the impetus given by the new ideas. In the late Sixties and early Seventies, Pasteur made his great discoveries. These were followed by the work of Koch and others in the Eighties, and the science of bacteriology soon arose. The experimental method was at once applied to the field of sanitation, including water purification. Although filtration had been used for a part of the London supply as early as 1829, the benefits of the process were not appreciated, nor its action fully understood for many years after that date. Following a cholera epidemic in 1839,

* Cambridge, Mass.

† Long unavailable, but now reprinted in "State Sanitation", Harvard University Press.

filtration was extended rapidly, and, in 1855, was made compulsory for the London Metropolitan District. It was the rise of the science of bacteriology and all that went with it which caused the modern development of this greatest of the sanitary arts—the art of water purification.

Fifty years ago the quality of the existing water supplies in the United States was low, judged by modern standards. Clearness and freedom from color, taste, and odor were the ruling standards, and even these were often not complied with. In some regions a great deal of attention was being given to lead poisoning and to hardness. Water analysis was confined to the mineral constituents. The germ theory of the transmission of disease through the agency of sewage polluted water had not arisen. In the United States, water purification was practically an unknown art.

Beginnings of Filtration in the United States.—In 1866 the late James B. Kirkwood, Past-President, Am. Soc. C. E., was sent to Europe by the city authorities of St. Louis, Mo., to study the new methods which had been coming into use in England and Germany, for besides the London filters there were, at this time, filters in many of the English, and in most of the large German, cities. Kirkwood's report, the first important document on filtration published in this country, may be said to mark the beginning of the history of water purification in America. His plan for filtering the muddy water of the Mississippi River, which was based on his European observations, was not adopted by the municipal authorities of St. Louis. It was fortunate that the plant was not built, for later studies of the filtration of such waters have shown that it certainly would have failed. Several other filters, however, were built as a result of his report, notably the filter at Poughkeepsie, N. Y., which was constructed in 1873 to clarify the water of the Hudson River. This filter was the most successful of several which were built about that time.

In 1878, Professor William Ripley Nichols went to Europe to study water purification for the Massachusetts State Board of Health and his report was printed in the Annual Report of the Board of that year. Five years later, his notable book on "Water Supply" was published, in which he discussed not only the problem of filtration, but that of tastes and odors as related to the algæ. In many ways, this book was a pioneer in its field. For a considerable time the center of interest in matters connected with water purification was in Massachusetts. In 1887, the State Board of Health established an experiment station at Lawrence, and the purification of water as well as the treatment of sewage was studied from all possible angles, engineers, chemists, and biologists contributing to the investigations. The experiments were made on the relatively clear but polluted water of the Merrimac River. Some of the men prominent in connection with this work were the late Hiram F. Mills, Hon. M. Am. Soc. C. E., the late Frederic P. Stearns, Past-President, Am. Soc. C. E., Allen Hazen and George W. Fuller, Members, Am. Soc. C. E., H. W. Clark, Edwin O. Jordan, Mrs. Ellen H. Richards, Professor Thomas M. Drown, and Professor William T. Sedgwick, the last two acting in an advisory capacity. This was the first important scientific study of the subject of water purification in America, and its results did much to develop the art.

While these experiments were under way, a notable epidemic of typhoid fever swept down the Merrimac Valley and included the City of Lawrence which took its water supply from the river. It soon became evident that a filter was needed, and, in 1893, a sand filter designed by Mr. Mills was put into operation. This filter was built as a single bed without a roof and involved certain complicated arrangements of coarse and fine sand, ideas which have not been followed in other plants; in fact, the filter has been largely rebuilt in recent years. This filter became a great object lesson to the country, as it showed conclusively that polluted water could be made safe for drinking. It was a practical demonstration of the ideas which had been developed at the Lawrence Experiment Station.

Mr. Hazen, after his Lawrence experience and after conducting experiments at the World's Fair in Chicago, Ill., spent a year in Europe studying foreign practice, and from this experience came his book on "Filtration", which for a generation was the leading authority on the subject. Unfortunately, it is now out of print.

Meanwhile, filtration had been developing in another direction, in regions of the country where the water supplies were muddy. In 1883 the late Charles Hermany, Past-President, Am. Soc. C. E., conducted some experiments in the filtration of the water supply at Louisville, Ky. In 1884, Alpheus Hyatt obtained a patent on the use of sulphate of alumina as a coagulant, and in the same year a mechanical filter using such a coagulant was built at Somerville, N. J. Although mechanical filters had previously been used for clarifying water for industrial purposes, this was the first application of the method to a public water supply. The name of Professor Albert R. Leeds should be remembered in connection with the early development of this process, as he was its real inventor. A mechanical filter was built for the water supply of New Orleans, La., but failed to meet the requirements and was removed at great loss to the promoting company. Several other installations were made in different parts of the country. Some of these early mechanical filters gave fairly good results, but others failed. It became evident that the process was not only useful but necessary for the purification of muddy waters, and also that engineers had not learned how to utilize it successfully. Then, Science came to the rescue. In 1893, the late Edmund B. Weston, M. Am. Soc. C. E., made some tests of mechanical filters for the City of Providence, R. I., which should be remembered as the first carefully conducted tests with this type of filtration. From 1895 to 1897, Mr. Hermany again studied the problem of filtration, this time with the assistance of Mr. George W. Fuller and a corps of engineers, chemists, and bacteriologists, acting for the Louisville Water Company and several companies interested in the construction of mechanical filters. These experiments, conducted on the water of the Ohio River, resulted in a notable report published by Mr. Fuller. The Louisville experiments were devoted to a study of the theory of the filtration of clay-bearing waters and the use of coagulation and sedimentation in connection with mechanical filtration. The principles thus established have served as the basis of the mechanical filters which have

since been constructed in the United States. Various devices used in mechanical filtration were also tested at Louisville and their weaknesses pointed out.

The experiments at Lawrence and Louisville demonstrated the fact that different kinds of waters require different methods of purification; that although slow sand filtration can be used successfully with a relatively clear water like that of the Merrimac River, at Lawrence, coagulation must accompany filtration in the case of muddy waters, such as those of the Ohio and Mississippi Rivers. With this thought in mind there followed a series of tests in different parts of the country. At Pittsburgh, Pa., Mr. Hazen conducted experiments to compare the relative advantages of sand and mechanical filtration for the water of the Allegheny River, and his report thereon was published in 1899. Mr. Fuller made further experiments in mechanical filtration in Cincinnati, Ohio, in 1898, and experiments were also made at Washington, D. C., Superior, Wis., New Orleans, La., Philadelphia, Pa., Reading, Pa., Boston, and elsewhere. It was a time of experimentation, and much of this experimental work was done by men who had received their early training in Massachusetts.

Progress of Filtration.—The first large slow sand filter to be constructed after that built at Lawrence, was designed by Mr. Hazen for the City of Albany, N. Y., to treat the water of the Hudson River. This filter was put into service in 1899. It differed in several respects from that at Lawrence, especially by having a masonry cover, experience having already shown that cold weather interfered with the operation of open filters like those at Poughkeepsie and Lawrence. The Albany filter has served in many ways as a model for the sand filters which have since been constructed in the United States, such as those at Washington, D. C., Philadelphia, Pa., Providence, R. I., and many other cities.

The first important mechanical filter to be constructed along modern lines was that of the East Jersey Water Company, at Little Falls, N. J., which was put into service in 1902 to purify the water of the Passaic River. As long as the basic Hyatt patent was in force the construction of mechanical filters had been confined chiefly to those built by private companies; after the expiration of this patent in 1901, however, the art of mechanical filtration advanced rapidly. In 1904, a large mechanical filter was built for the Hackensack Water Company in New Jersey, and many others followed.

It will be seen, therefore, that it was just about the beginning of the 20th Century when water purification in the United States really began to advance as a practical measure. The last decade of the 19th Century had been one of experimentation. The work done by the chemists and bacteriologists was to be taken up by the engineers and pushed rapidly.

Since 1900, there has been a great increase in the use of filtration. In 1870, practically no filtered water was in use in this country. In 1880, 30 000 people in cities having populations of more than 2 500 were using filtered water. In 1890, this number was increased to 310 000; in 1900, to 1 860 000; in 1910, to 10 805 000; and, in 1920, to at least 20 000 000. At the present time, more than one-third of the people living in cities which contain

2 500 or more inhabitants are using water which has been filtered. The number of filters in the country is probably not far from 800.

Present Problems.—Water purification would not be a growing art unless there was new work ahead, and unless there were new problems being studied. A survey of the situation reveals some interesting facts and tendencies.

In the first place, not all the water supplies of the country are being adequately protected. There are still some places where grossly polluted water is being distributed to the people, although year by year these places are decreasing in number. There are many places where the water is reasonably safe, but is not attractive at all times for one reason or another. Fifty years ago, and even twenty-five years ago, the New England water supplies were on the whole better in quality than those in other parts of the country, but to-day filtration has become so common in regions where the streams are naturally muddy that the New England supplies, few of which are filtered, suffer in comparison with filtered waters elsewhere, in so far as the quality of attractiveness is concerned. In New England, the policy of utilizing natural storage and preventing pollution in all possible ways was wisely adopted, but these measures have their limits and, with the increasing populations, the filtration of surface waters is likely to become universal; it may even be required by law.

There are many places in the United States where disinfection has been adopted without filtration or in place of adequate attempts to prevent pollution. Gradually, the weakness of this policy is being discovered. Disinfection is very cheap, but cheap disinfection does not accomplish what filtration does, and it should not be accepted as a substitute for vigilance in protecting a water supply against pollution.

Since the typhoid fever rates of the country have been greatly reduced, consumers are thinking less about the dangers of their water supplies and more about its physical appearance, about its corrosive properties, and about its fitness for industrial uses, that is, they are seeking refinements. Thus, a new class of problems, such as water softening with the use of lime and soda, or with permutit, the removal of iron and manganese, and the neutralization of acidity, is receiving the attention of water chemists.

Many of the filters built 10 and 20 years ago have been in use long enough to show defects in practical operation, and experience has indicated wherein changes can be made to avoid depreciation, reduce repair costs, save money in operation, and get better results. Filter designs are slowly being modified as a result of these findings. It is natural that at the time of building these filters officials should give the greatest attention to construction cost, but, as time goes on, it is the costs of operation which are scrutinized. The usual lower first cost of mechanical filters is a reason why it is sometimes easier to induce cities to construct that type of filter rather than a slow sand filter which usually costs more to build but less to operate. Higher operating costs, however, make mechanical filters less popular after a term of years. The economics of filter operation can now be studied more carefully than was possible twenty-five years ago, and this is a phase of the subject which is likely to receive increasing attention.

Methods of testing the performance of filters by the use of analyses have not changed greatly in twenty-five years. In the early days, stress was laid on the percentage removal of bacteria, color, turbidity, and organic matter. Tests were made to show what the filter was able to do. Engineers now are more interested in knowing what the filter actually does, and the analyses must serve as a test of the filter operator as well as the filter itself. When talking about water analyses, Dr. Drown used to say that "a state of change is a state of danger." Now, the principle is emphasized that a state of irregularity is a state of danger. Filters are wanted which operate so as to give a uniform product from day to day and hour to hour. Therefore, attention is being given to the application of the mathematical theories of probability to filter records. There are great possibilities in this study, but they are appreciated at present by only a few engineers.

At the moment, a new impetus is being given to many of the problems of water purification by the use of new and simple methods of determining the true acidity of water in terms of the hydrogen ion concentration. This and other methods of physical chemistry are likely to clarify many ideas in regard to problems of coagulation, corrosion, disinfection, and the growth of algæ.

Bacteriological tests of water have never been as definite as chemical tests. The difficulties have been partly analytical and partly statistical. The test for *B. coli* has been largely, and, on the whole, successfully, used; but it has many shortcomings. The use of tests for determining the presence of spore-forming organisms is now under discussion, so that water bacteriologists as well as water chemists are active in their researches.

In the field of vital statistics new ideas are also coming to the front. The typhoid fever death rates are becoming so low that they can no longer be regarded as sufficient to measure the healthfulness of a water supply. The so-called Mills-Reincke theorem, which held that for every death from water-borne typhoid fever there were several deaths from other diseases due to water, has been found not to hold generally, although there is much truth in it. Polluted water may cause sickness of one kind or another, which does not find record in the vital statistics of the community. Some more careful measure of the effect of water on the health of the community is urgently needed.

Thus, it is seen that there are activities among engineers, chemists, biologists, and statisticians in the field of water purification, a fact which is full of promise.

RECENT DEVELOPMENTS IN WATER FILTRATION

By Allen Hazen,* M. Am. Soc. C. E.

A Dutch author recently classified all waters in public water supplies in two classes, namely, aggressive and quiet. He defined aggressive waters as those having a tendency to attack and corrode the iron pipes through which they flow and other metal fixtures with which they come in contact. Quiet waters do not have this tendency to attack metals.

Aggressive water is very troublesome in a distribution system. The treatment of water so as to purify it adequately without making it aggressive is one of the most difficult problems at present. Water leaving the filter may be of good color, show a good bacterial test, be free from turbidity and other injurious elements, and it may appear to be in all respects suitable for public water supply, but if it is aggressive it will not be satisfactory to the users. An aggressive water attacks and tuberculates the pipes, reduces their carrying capacity, and will ultimately destroy them. In doing this, however, some of the iron of the pipes is taken up, and it is this iron which makes the water objectionable. The iron taken up causes a supplementary coagulation in the water, and the floc formed in this way settles out where the velocity of flow is low. This deposit is a dirty mud which remains on the bottom of the pipes until the flow is increased sufficiently to move it. It then moves forward with the flow and makes the water dirty and disagreeable, giving just cause for complaint.

There is another condition similar to this, which is frequently associated with it. Water which has received chemical treatment before filtration, frequently passes the filter before the chemical reactions are altogether complete. Such water leaving the filter contains a small quantity of coagulant which separates in the pipes and produces the same conditions as those produced by the iron taken up from the pipes. Probably this condition of incomplete chemical reaction at the time of filtration is found mainly with aggressive waters. At any rate, the two conditions produce similar results and frequently they are found together, in which case, the resulting conditions are much worse than they would be otherwise.

The question of why some waters are aggressive and others quiet is presented. Twenty years ago, chemists would have stated that it was, first, a matter of alkalinity, and carbonic acid would have been mentioned as a second important contributing factor. Alkalinity and carbonic acid really have something to do with aggressive qualities, but all the conditions cannot be accounted for by them. Other explanations must be found. Recently, new methods of testing have been originated. The theory of "ions" has been developed, and it is thought by some who have been studying the matter, that a connection between the "ions" and the aggressive qualities of waters can be traced. Tests based on these new ideas have been devised, and have been applied for the first time quite recently, and at present there seems to be some hope of a method of ascertaining more definitely when water will be quiet and when it will be aggressive. Knowledge comes first, and when engineers under-

* New York City.

stand better why waters are aggressive, it will be easier to find methods of treating them so as to keep them quiet.

At present, a great deal is known in a practical way as to which waters are aggressive and give trouble because of their aggressiveness. The waters of the Great Lakes are almost always quiet and can be filtered and purified in a manner to keep them so. In the Mississippi Valley, from the Alleghanies to the Rockies, the surface waters used for public water supply are generally hard, and can be handled, by carefully managed treatments, so that the resulting products will be reasonably quiet. Thus, through all that broad expanse of country the problem of aggressive waters is much less important and difficult.

Soft waters are frequent nearer the coasts, both East and West, and they often carry organic matters of vegetable origin. These waters are much more apt to be aggressive, and it is with them that the greatest practical difficulties have been met. Experience shows that it is difficult to treat some of these waters by methods otherwise effective, which will clear them up and make them satisfactory for public water supplies, and which will leave them quiet. Many of these waters require chemical treatment, and no other means of adequately purifying them has been found, but in some way this chemical treatment stirs up the sleeping ingredients of the water and makes them aggressive.

Decolorizing yellow swamp waters by means of chemicals seems particularly difficult of accomplishment without bringing out the aggressive qualities. The question of color is mainly one of esthetics, and it may fairly be asked whether the increase in aggressive qualities in decolorizing water does not largely or entirely offset the advantage gained, and whether it would not be wiser in some cases to get along with less decolorization.

The chemical treatment of water introduced many years ago had numerous advantages and came to be widely used. Many waters, however, can be treated with entire success without chemical treatment. This depends on the character of the water, and on what is to be removed from it. Chemical treatments may have been used in some cases where it would have been better to have avoided them. Decision to use them may have rested on consideration of all the other factors in the case. When the development of aggressive qualities is also taken into account, perhaps, in some cases, the decision would have been better made the other way.

It may be mentioned that one industrial establishment is now constructing a plant for purification of water without the use of chemicals. Formerly, chemically treated water was used, and experience with it has indicated the difficulty of furnishing water sufficiently quiet so that it would not damage the delicate products of this particular establishment.

The question of arranging all the treatments of public water supplies so as to keep them always quiet is one of the most interesting and important aspects of water purification at the present time.

INTERFERENCE WITH WATER FILTRATION PLANT OPERATION BY WASTES FROM BY-PRODUCT COKE OVENS AND GAS-WORKS.

By C. A. Emerson, Jr.,* M. Am. Soc. C. E.

The discharge of liquid wastes from by-product coke ovens and commercial gas plants into streams and lakes from which water supplies are taken, has become, during the past few years, a matter of grave concern to many medium sized and several of the largest cities, due to the disagreeable tastes and odors which these wastes impart to the water.

As the superiority of by-product coke ovens over the old beehive type is now fully established, it follows that the number and capacity of these installations will undoubtedly increase. By-product ovens of necessity must be located reasonably close to a steel works or a large municipality in order to secure a ready market for the gas given off during the coking of the coal, and, therefore, it is almost certain that interference with surface water supplies will also increase, if proper precautions are not taken.

The wastes from the tar separators, the ammonia, benzol, and naphthalene stills of by-product coke ovens, together with the sludge removed from the various gas and oil mains throughout the plant, impart a characteristic taste to river water. This taste may persist for days and, after transportation for many miles, be noticeable in the effluent from well operated, water filtration plants of modern types, taking their raw water supply from the stream. The application of chlorine, even in the minute quantities used for water supply treatment, seems to accentuate rather than diminish these taste-producing characteristics. Under favorable conditions, dilutions as great as several million parts of chlorinated water to one of the wastes have failed to cause complete disappearance of this taste.

Fortunately, the remedy for this pollution lies within the works, and consists simply of evaporation of the contaminated liquid wastes by using them for quenching or cooling the glowing coke taken from the ovens. The waste water which is not turned into a cloud of steam, but passes through the quenching platform, is intercepted in settling basins and again used for quenching. In all the plants coming under the speaker's observation, the water requirements for coke quenching have exceeded the volume of contaminated wastes, so that this remedy has not worked a hardship on the owners.

That the remedy is effective is evidenced by the fact that the largest by-product coke oven installation in the world is operating on the Monongahela River, ten miles above the intake of a municipal water-works, without causing contamination of the supply. Take another stream in Pennsylvania as an example: The installation of appliances for using liquid wastes from stills for quenching the coke silenced complaints among the consumers of filtered water in several municipalities along the stream about seventy miles below the works. That the method is beyond the experimental stage is indicated by the fact that it has been in successful use at numerous plants for several years and that, in

* Harrisburg, Pa.

some instances, the volume of liquid wastes evaporated is in excess of 1 000 000 gal. daily.

Interference with water supplies by the wastes from plants producing water gas for illuminating and power purposes has been noticeable in different parts of the United States. Definite evidence that the tastes due to these wastes are as persistent, or that they can be detected in as high dilutions, as those from by-product coke ovens, seems to be lacking. There is, however, sufficient evidence to justify the statement that the raw wastes from these plants, when subjected to dilutions available in rivers of considerable size, will cause tastes after transportation by the stream for several miles. With these wastes, also, the taste-producing characteristics are accentuated by chlorination of the drinking water taken from the stream.

Aeration of the raw water, thus contaminated, prior to filtration seems to be of considerable value, but in this case, again, a certain and effective remedy can be developed at the gas-works. It has been found that installation of settling and skimming basins to permit the removal of the greater portions of the heavier tars and the light floating oils, with the circulation of the basin effluent for re-use in the "gas scrubbers", will not only materially decrease the proportion of active taste-producing constituents, but will reduce the volume of wastes reaching the stream to a small fraction of that which occurred before the water was re-circulated through the plant. In addition, the company recovers marketable materials to offset the cost of operating the circulating pumps.

In one instance coming to the speaker's attention recently, it was deemed advisable to require the gas company to go even further, and to keep all liquid wastes from the stream. This company, by the use of settling basins and circulating pumps, had decreased the volume of wastes from 50 000 to about 5 000 gal. per day. This waste was high in turbidity, had an oily appearance, and was brown in color, but it was found that after straining it through a layer of coke breeze, about 6 or 8 in. in thickness, and receiving a moderate application of ferrous sulphate solution, it could be passed through a small sand filter at a rate of about 400 000 gal. per acre daily and then become a satisfactory boiler water. Contrary to all expectations, the sand filters did not clog immediately, but were operated for runs of several days' duration between scrapings.

Occasionally, there has been doubt as to legal authority to require abatement of the class of pollutions described, as the laws controlling stream pollution due to the discharge of industrial wastes are weak and vague in many States; however, instances are recorded where applications for restraining injunctions have been granted under the broad general rights of lower riparian ownership and, accordingly, it is believed that in a surprisingly large number of cases definite obligations can be imposed on owners of such works to prevent troublesome stream pollution.

RECENT PROGRESS IN THE REDUCTION OF THE TYPHOID DEATH RATE AND ITS SIGNIFICANCE.

By C.-E. A. Winslow,* Esq.

It seems worth while in a symposium of this kind to call attention to the remarkable progress which has been made in the reduction of the typhoid death rate since George A. Johnson, M. Am. Soc. C. E., presented his admirable review of "The Typhoid Toll" before the American Waterworks Association, five years ago.† One may go back perhaps even a little farther with profit and take as a basis for comparison the statistics for certain large cities for the period, 1898–1908, presented by Mr. G. R. Taylor.

TABLE 1.—Typhoid Statistics of Cities of the United States, 1898-1908 and 1917-1919.

City and State.	Typhoid death rate per 100 000 population. Average. 1898–1908*.	Typhoid death rate per 100 000 population for 1917, 1918, 1919 (United States mortality statistics):		
		1917.	1918.	1919.
Worcester, Mass	15.3	5.8	3.4	2.8
Fall River, Mass.	15.7	18.3	15.0	3.3
St. Paul, Minn	16.3	2 6	3.5	3.4
New York, N. Y	17.6	4.2	3.6	2.2
Rochester, N. Y	18.5	3.0	1.8	3.4
Jersey City, N. J	18.5	3.8	5.5	2.4
Newark, N. J	19.2	4.3	4.5	2.2
Syracuse, N. Y	19.2	6.1	9 0	7.1
Providence, R. I.	19 5	6.8	5.1	3.8
Milwaukee, Wis	19.7	6.2	6.3	3.7
Detroit, Mich	20.9	12.7	7.9	5.2
Paterson, N. J	20.9	12.8	4.5	3.7
Omaha, Nebr	23.1	4.4	5 9	5.3
Boston, Mass	23.3	3.1	2 6	2.4
Chicago, Ill	25.3	1.9	1 6	1.3
Buffalo, N. Y	27.0	9.9	7.7	6.6
St. Louis, Mo	27.7	8.4	7.9	6.6
Scranton, Pa	28.8	6.6	6.6	1.5
Minneapolis, Minn	34.9	6.7	8.4	3.2
Baltimore, Md	35.5	14.4	11.5	8.2
Toledo, Ohio	35.8	8.0	11.2	4.2
Los Angéles, Cal	36.8	6.0	3.9	4.8
Memphis, Tenn	38.7	26.1	16.2	62.8
Indianapolis, Ind	39.3	10.5	6.6	4.5
New Haven, Conn	40.5	10.3	4.4	5 6
San Francisco, Cal	42.0	4.5	4.2	3.0
Kansas City, Mo	43.0	12.1	15.0	10.9
New Orleans, La	43.7	23.5	20.0	18.2
Denver, Colo	48.8	5.7	9.2	3.5
Cincinnati, Ohio,	50.7	4.0	4.8	3.0
Philadelphia, Pa	53.6	6.3	4.9	4.3
Washington, D. C	55.5	12.2	11.6	3.7
Louisville, Ky	57.4	16.3	16.3	10.7
Columbus, Ohio	57.5	7.6	8.7	3.0
Pittsburgh, Pa	116.9	12.2	10.7	6.5

* "A Classification and Study of the Typhoid Statistics of the Cities of the United States", by G. R. Taylor.

The statistics for this period are compared with those for 1917, 1918, and 1919, in Table 1, and Table 2, shows the distribution of these cities in the two periods, by their grouping in regard to typhoid mortality. It will be

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† *Journal*, Am. Water Works Assoc., Vol. 3 (1916), p. 249.

noted that in 1898-1908, each of the thirty-five cities had a death rate of more than 15 per 100 000, and that seventeen, or practically one-half of them had a rate of 30 or more. In the last three years, however, only one city of the same thirty-five has had rates of more than 15, only six have had rates of more than 10, while more than one-third have had rates below 5 per 100 000. This is a truly remarkable achievement and one which is not confined to the cities in question, but which is general throughout the United States.

TABLE 2.—DEATH RATE FROM TYPHOID FEVER IN CERTAIN LARGE CITIES OF THE UNITED STATES.

Distributed by Classes.

Death rate per 100 000.	Under 5.	5-9.9.	10-14.9.	15-19.9.	20-29.9.	30-39.9.	40-49.9.	50-99.9.	100+
Number of cities in each class, 1898-1908.........	0	0	0	10	8	6	5	5	1
Number of cities in each class, 1917-1919.........	12	17	4	1	1	....	....	....	....

In Table 3 are presented the data for the gradually expanding area of the registration States, which show that for this unit of area the typhoid rate has dropped in thirty years to less than one-third of its 1890 value. The actual accomplishment is even more striking than such a tabulation would indicate for, as the registration area has increased, it has tended to include more and more cities with relatively primitive sanitary organization. Table 4 shows the figures for the registration States as the area existed in 1900, and it will be seen that when the same States are compared, which had a rate of 25.4 in 1900, the rate had fallen to 6.5 in 1916-19, a reduction of nearly 75 per cent.

TABLE 3.—DEATH RATE PER 100 000 POPULATION FROM TYPHOID FEVER IN THE EXPANDING REGISTRATION STATES FOR DIFFERENT PERIODS, BY CITIES AND BY RURAL AREAS, 1890–1919.

Place.	PERIOD CONSIDERED:					
	1890.	1900.	1901-05.	1906-10.	1911-15.	1916-19.
Registration States..........	36.0	25.4	24.9	24.6	16.3	11.9
Cities........................	39.0	25.3	24.5	25.3	14.8*	9.2*
Rural areas	31.4	25.5	25.4	23.8	17.8	13.7

*Municipalities of 10 000 or more inhabitants in 1910.

Such results are highly gratifying, but the speaker would like to utter a word of caution relative to the claims any one may be inclined to make in regard to the factors which have brought about this reduction. One is naturally tempted, when the figures for such cities as Pittsburgh, Pa., are

examined and where the reduction in the typhoid death rate was from 116.9 (1898-1908) to 6.5 (1919), to claim all the credit for the water-works engineer, but a little consideration will show that there are other powerful forces at work.

TABLE 4.—DEATH RATE PER 100 000 POPULATION FROM TYPHOID FEVER IN THE ORIGINAL REGISTRATION STATES FOR DIFFERENT PERIODS, BY CITIES AND BY RURAL AREAS, 1900–1919.

Place.	PERIOD CONSIDERED:		
	1900.	1911-15.	1916-19.
Original Registration States †..	25.4	11.1	6.5
Cities..............................	25.3	13.0*	6.9*
Rural areas......................	25.5	8.4	5.8

* Municipalities of 10 000 or more inhabitants in 1910.

† Connecticut, District of Columbia, Maine, Massachusetts, Michigan, New Hampshire, New Jersey, New York, Rhode Island, Vermont.

The data for the expanding registration area presented in Table 3 would suggest that the typhoid death rate has fallen much faster in cities than in rural areas, but this conclusion is wholly fallacious, for the expanding registration area has tended to increase the proportion of sub-standard rural areas much faster than the proportion of sub-standard urban areas. If the rural and urban changes in the death rate in the original registration States of 1900 are compared, it will be found that the reverse condition obtains; in other words, when the same group of cities is considered, the rural death rate has decreased more markedly than the death rate in urban communities. This fact alone is sufficient to indicate the complex nature of the problem, since improvements in water supplies have certainly been vastly more effective in cities than in rural districts. The improvement in privy sanitation, the war against the fly, the pasteurization of milk, and, since the World War, the use of typhoid vaccine, have each played an important part in the conquest of typhoid fever, and there is glory enough for all in the fact that at last the typhoid statistics of the United States can be compared with those of Europe without the sensation of humiliation.

THE EFFECT OF WATER PURIFICATION AND IMPROVEMENTS IN WATER SUPPLIES ON THE TYPHOID FEVER DEATH RATE IN NEW YORK STATE.

By C. A. Holmquist,* Esq.

Before discussing the reduction of typhoid fever, due to improvements in water supplies, the speaker will review briefly the part played by the New York State Department of Health, more particularly the Division of Sanitary Engineering, now the Division of Sanitation, in improving the public water supplies of the State through its general supervision over their sanitary quality. The first State Board of Health of New York was organized in 1880, under the provisions of the Public Health Law. This Board consisted of nine members, nearly all of whom were physicians.

In 1901, the State Board of Health was abolished, and a State Department of Health, with a commissioner at its head, was created. In 1906, the Division of Sanitary Engineering with Theodore Horton, M. Am. Soc. C. E., as Chief Engineer, was established. Up to that time, relatively little attention had been paid to public water supplies. Engineers were employed from time to time to advise the Board and to investigate matters pertaining to sewerage and sewage disposal, stream pollution, and other sanitary engineering problems, and although rules and regulations had been enacted for the protection of certain surface supplies from contamination, no comprehensive investigations had been made to determine the conditions of the water supplies in the State, or to improve them.

Since the establishment of the Division of Sanitary Engineering in 1906, one of the principal, if not the most important, functions of the Division has been the supervision over the sanitary quality of public water supplies. Of about 550 public water supplies in the State all have been investigated and reported on at least once and many of them a number of times.

Rules and regulations for the protection of about ninety surface supplies have also been enacted. Since 1906, the number of persons served by public water supplies in New York State has increased from about 6 000 000 to 8 500 000, and the number of persons supplied with purified or treated water has increased from 700 000 to nearly 7 000 000.

The results of these improvements are clearly reflected in a decrease of the typhoid fever death rate in the State as a whole. Before 1906, the average death rate from typhoid fever for the whole State, including New York City, was 23.4. Since 1906, the average rate has been 9.4, a reduction of 14 per 100 000. It will be noted also that the rate has decreased each year with marked regularity, until 1920 a remarkably low level of 3.5 was reached, a figure which about 20 years ago was generally considered to be the probable irreducible minimum for the United States. In all probability, the rate will be lower, but the decrease necessarily will be less marked in the future than in the past. The figures available up to the end of September, 1921, indicate that the rate for this year will be slightly less than those for 1920.

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Although other factors, such as improvements in milk supplies, etc., no doubt have had some effect in decreasing the typhoid fever death rate, the speaker believes that the decrease has been due largely to improvements in water supplies.

The municipal authorities of Cohoes, N. Y., constructed filters and commenced sterilizing the water supply in 1911. Between 1900 and 1911, the average death rate from typhoid fever was 85.5, fluctuating from a minimum of about 58 to a maximum of about 133. Since the filters were built, the average death rate has been 17.3 and, for two years, no deaths from typhoid occurred in that city.

Filtration and sterilization plants were constructed in Niagara Falls, N. Y., in 1911. Before that time, the average death rate from typhoid fever was 131.8; since the establishment of filters the average death rate has been 9.4.

Albany had an average typhoid fever death rate of 89.4 per 100 000 before the construction of its filter plant in 1899. Since the filters were built, the average death rate from typhoid fever has been 17.5, and it has been gradually decreasing until in 1920 the rate was about 3 per 100 000.

At Binghamton, N. Y., the average death rate from typhoid before the filters were built, was 56.2, and the average death rate from typhoid since the construction of the filters has been 11.7.

It is evident that in each case a decided drop in the typhoid fever death rate occurred immediately after the construction of the filtration plants. It is also evident that the city, the water supply of which was most grossly polluted before purification, had the highest typhoid rate and the one the water supply of which received the least pollution had the lowest rate. The distance which the pollution has to travel and the velocity of the stream are, of course, important factors in the case of water supplies derived from rivers.

For Niagara Falls, which had the highest death rate of any city in the State before the construction of filters, the water supply is taken from the Niagara River. Buffalo, N. Y., with a population of about 500 000, discharges its untreated sewage into the river about 15 miles above the intake of the Niagara Falls water supply, and the raw sewage from the Cities of Tonawanda and North Tonawanda, N. Y., with a total population of 25 000, is discharged into the river about 10 miles above Niagara Falls. With the sewage of 500 000 people discharging into a comparatively swift stream at a relatively short distance above the water supply intake, it is not surprising that Niagara Falls had a high typhoid fever rate before water purification was provided.

The typhoid death rates at Cohoes and Albany, before the construction of filters were nearly the same, and the raw water is of a somewhat similar character. Cohoes obtains its water from the Mohawk River about 18 miles below where the sewage from Schenectady, with a population of about 90 000, is discharged into the river, formerly without any treatment whatever.

Albany obtains its water supply from the Hudson River about 8 miles below the mouth of the Mohawk River, and the intake is from 4 to 10 miles below Troy, Cohoes, and Watervliet, which have a combined population of

110 000. The sewage from all these cities is discharged directly into the Hudson River without treatment.

The Susquehanna River, from which the City of Binghamton obtains its water supply, receives comparatively little raw sewage within 50 miles of the intake. The typhoid fever death rate at Binghamton, although high, was nevertheless much lower than at either of the other cities referred to.

In closing, the speaker wishes to point out that to Mr. Horton, who was Chief Engineer of the State Department of Health from 1906 to July 1st, 1921, is due in a large measure the credit for the improvement of public water supplies in this State.

PURIFICATION OF SOFT COLORED WATERS

By Robert Spurr Weston,* M. Am. Soc. C. E.

Engineers engaged in water supply work are in substantial agreement that a drinking water, to satisfy critical modern consumers, should have an average color of not more than 10 parts per million on the platinum scale, and its maximum color should not exceed 15 parts. Supplies from ground sources and from many lakes and ponds conform to this standard, but most of the clearer surface waters possess a brownish coloration, well above the limit of 15 demanded by the modern consumer. In Massachusetts, according to the reports of the State Department of Health, practically all the ground-water supplies which are free from iron, have colors not exceeding 10, while of 86 surface supplies, 50 or 58% have average colors in excess of this standard. The appearance of these colored waters draws unfavorable comment from Western visitors who are accustomed to water filtered clear and colorless, from turbid streams.

A growing appreciation of the esthetic characteristics of drinking water is leading to the betterment of these colored supplies by purification, the current methods being storage and filtration. Storage for months or years in good reservoirs effects a decolorization of water. In this, iron plays an important part in flocculating minute particles of suspended matter which constitute what is called color. Several examples of this were given by Ralph H. Stearns,† Assoc. M. Am. Soc. C. E., which show reductions of from 4 to 69% through storage. It is most often impracticable to store waters long enough to decolorize them sufficiently to compare them, with a reasonable degree of favor, with filtered waters and the average ground water, and, for this reason, as well as further to safeguard the public health, many stored colored waters are filtered. This practice is usual abroad, and is coming into increasing favor here, as evidenced by the new purification plant at Hartford, Conn., and the plans for the additional supplies for New York City, and Providence, R. I. The first large filters were built at Cleveland, Ohio, and Somerville, N. J., and the second one at Washington, D. C. The filters at Hartford are of the slow type, and although the color of the effluent will probably not average as low as 10 parts per million, a supply of reasonably good appearance is assured by this method. Of course, color removal by slow filters has been the subject of many discussions, and the statements made have rarely taken into account the physical and chemical nature of color.

Most of the color in water is in the form of a colloidal suspension, that is, the particles are so fine and so dispersed in water that they remain in suspension and act like true solutions, except that the Tyndall ray and the ultra-microscope reveal them. These colloidal particles are not all alike. They possess electrical properties, and when electrolyzed some of the particles will migrate to the positive, some to the negative pole. A small part of the color is apparently in true solution and is not affected by the electric current. It is reasonable to believe that colored water from a large lake or reservoir

* Boston, Mass.

† *Journal*, New England Water Works Assoc., Vol. 30, p. 20.

will contain finer particles of color than a river water. This belief is supported by the higher color removal by sand filters supplied by river than by reservoir waters. A slow sand filter will remove one-third of the color, but when treating certain river waters such filters may remove more than one-half the color, although they are usually unable to remove more than one-fifth of the color from a stored water.

There are waters, therefore, which require chemical treatment prior to filtration. In nearly all such cases, sulphate of alumina with or without an alkali has been used. The dosing has been based on the appearance and alkalinity of the water, and the results, especially in small plants operating without laboratory control, leave much to be desired. Recently, George C. Whipple, M. Am. Soc. C. E., has stated:

> "The use of alum with short periods of coagulation and mechanical filtration of the ordinary type is, in my opinion, inappropriate to a soft colored Massachusetts surface water. * * * * The corrosion problem in our State is serious and must not be made more so by inappropriate chemical treatment."

The mechanical filter, *per se,* is an efficient device, and the remedy lies in better chemical treatment. If the waters could be treated more scientifically, the period of coagulation might be reduced and the rate of filtration increased. Better treatment, however, must await the application of modern chemical and physical theories to practice, and this will require much investigation and many trials before any rules can be written for the non-technical operator.

At the present time, sulphate of alumina is added until coagulation takes place, and, in most cases, soda is added to maintain an alkalinity of at least 7 parts per million in the water delivered to the mains. Frequently, this method fails. At some places, such as Warren, R. I., soda cannot be used at certain times of the year, and the alkalinity is maintained by the addition of calcium carbonate in the form of powdered chalk. If sulphate of alumina is added to excess, better decolorization will be obtained, but the treated water will corrode the distribution system. In the one case, the soda has been added to the filtered water. At Wilmington, N. C., the water is first overdosed with alum, and after it has passed through the coagulating basin, it is dosed with enough soda or lime to insure a final alkalinity of 10. In these cases, both the optimum decolorization by sulphate of alumina and the inhibition of corrosive action is attained. At Exeter, N. H., it was impossible, even by adding sulphate of alumina to excess, to reduce the average color much below 20 parts per million. Apparently, the water contained a large quantity of positively charged coloring matter which was not affected by aluminum hydrate having the same charge. By pre-treating the water with 0.5 parts of chlorine per million, a condition was brought about under which treatment with a small dose of sulphate of alumina easily reduces the color below 10 parts per million.

Ordinarily, one believes that if an equivalent quantity of soda is added to a unit dose of sulphate of alumina, aluminum hydrate will be produced. Try this in some cases in practice, and what results—a re-solution of the color.

In some cases reduce the equivalent dose of soda to, say, one-half, and a good decolorization may result. These facts are known, but the principles on which they are based are not yet clear.

There is also the problem of after-precipitation. In at least four rapid filter plants supplying New England towns, some of the aluminum hydrate passes the filters in a colloidal form to coagulate later in the distribution system. This was the chief of the many objections which the late Hiram F. Mills, Hon. M. Am. Soc. C. E., raised against alum treatment. This fault is noticed even at plants with coagulating basins of relatively large capacities, although not so frequently as at plants where the chemicals are applied directly to the influents of pressure filters. The trouble occurs most frequently in cold weather when all chemical reactions are retarded.

From what has been said, it is obvious that water purification experts are trying to solve what is, perhaps, the most difficult water purification problem, without a knowledge of the factors which enter into it, and until they can elucidate these factors, the demonstration of the theories is impossible.

For guides, engineers have depended largely on the color and alkalinity determinations. What do these mean? Simply a measure of the intensity of the one and the combining power of the other. No regular determinations record the nature of the color, and although the brown of one water may appear to be similar to the brown of another, it is no sign that they will respond to the same treatment. Thorndike Saville, Assoc. M. Am. Soc. C. E.,* showed how varied were the components of color in water. His experiments demonstrated the ultra-microscopic character of the "suspensoids" and "emulsoids", to use the modern terms, which make up the bulk of color in water, and that these carry electrostatic charges, positive or negative, depending on, and varying with, the water. Usually, the color particles have a negative charge.

It is obvious that the difficulty of finding the charge of the color particles and the necessary treatment is not much facilitated by the ordinary water analysis. In practice, it is assumed that the color carries a negative charge, and aluminum hydrate is used with its positive charge, to neutralize and coagulate it. This treatment does not dispose of the positively charged color particles, except that these may be removed by the absorptive action of the precipitate produced by the negatively charged particles and the hydrate. Special treatment for these positively charged particles is necessary, and the substances added must not neutralize the effect of the positive aluminum hydrate. The good effect of chlorine at Exeter, and of carbon dioxide in the experiment of Mr. M. C. Whipple† in increasing the electro-negative character of the color and, consequently, the decolorizing power of a given dose of sulphate of alumina, leads one to hope that methods for determining the electrostatic charges of color particles will come into more common use and their results will indicate more rational treatment than that commonly used at the present time.

* *Journal*, New England Water Works Assoc., Vol. 31, p. 78, *et seq.*

† *Journal*, New England Water Works Assoc., Vol. 31, p. 116.

The alkalinity determination simply expresses the quantity of sulphuric acid in terms of calcium carbonate required to neutralize the water. It does little else. Practically all the colored waters supplied to towns are alkaline. Yet, how different are the treatments required by waters of about the same alkalinity and color. An equivalent of magnesium hydrate has the same combining power as an equivalent of caustic soda. The former is called a weak, the latter a strong, alkali, and these adjectives describe their effects. By determining the hydrogen ion concentration, these differences of potency can be measured. This determination is coming into use; it has already been made part of the daily routine at Baltimore, Md., and other places. Apparently, there is an optimum hydrogen ion concentration for each water, where coagulation is most nearly complete. It is not far from PH = 7.0. The hydrogen ion determination is not difficult, and the speaker believes that it may show what conditions of the dissolved salts in water, that is, the electrolytes, are most favorable for flocculation of the colloidal color, probably in many cases by suggesting treatment with other chemicals as well as with sulphate of alumina.

The problem, therefore, is to substitute a specific for a panacea treatment by a more accurate analysis of color and a more proper dosage with chemicals, also by the creation of the most favorable condition in the solution—the water—for the flocculation and precipitation of the neutralized colloidal color. This seems to be complicated, but the speaker believes it will be worked out to a satisfactory solution in practice. Indeed, the collection and study of the more refined electro-chemical data should explain the successes and failures in practice, and should help to rationalize the treatment of colored waters. On the practical side is the lure of a greatly reduced cost for chemicals, which should stimulate research in this important field.

THE OPERATION OF RESERVOIRS FOR WATER SUPPLY.

By Samuel A. Greeley,* M. Am. Soc. C. E.

The developments of storage for water supply throughout many parts of the Upper Mississippi Valley is frequently a relatively costly project. The river valleys are wide and flat, and the slope of the streams is moderate. Thus, at Decatur, Ill., to impound about 9 000 000 000 gal. to furnish an estimated minimum yield of 42 000 000 gal. per day, it is necessary to flood about 5.5 sq. miles and the total cost of the project is almost $2 000 000. Several other water-impounding projects which have come to the speaker's attention, have called for expenditures in excess of bond limitations and have been financed by popular subscription.

Furthermore, south of Wisconsin and Minnesota, ponds, lakes, and streams for recreation purposes are scarce and inaccessible and by no means as plentiful as they are throughout New England, for instance, and portions of New York State and New Jersey. For these reasons, among others, there is a strong disposition to permit the use of these reservoirs for recreation purposes, including bathing, boating, fishing, and the like. The most extreme case which has come to the speaker's attention is Lake Milton, in Ohio, which furnishes the water supply for Youngstown, Warren, Niles, and other small communities in Ohio. This reservoir covers about 2.65 sq. miles and impounds about 9 000 000 000 gal. There are a number of resorts about its shores, and bathing, boating, fishing, and picnicing are allowed. Below the dam, the water flows several miles in open channel to the intake of the filter plant, receiving pollution on the way, which masks the effect of reservoir use.

It should be noted especially, however, that many of the surface water supplies of the Upper Mississippi Valley are turbid and otherwise unsatisfactory without filtration, so that modern water filtration plants are, in general, in operation in the case of the larger projects. This circumstance tends to render officials and the public less sensitive to the possibilities of pollution at the source.

Earlier Discussions.—The use and maintenance of reservoirs has been under consideration from time to time since 1907, when a thorough discussion† was presented before the Society on the relative value of the control of drainage areas and the installation of water filtration plants for the protection of public water supplies. The discussion at that time, however, did not relate primarily to the use of reservoirs as this use affected a safe load on water filtration plants.

A later, and a very able, discussion was presented in 1920, by X. H. Goodnough, M. Am. Soc. C. E., before the New England Water Works Association, deprecating the use of reservoirs for fishing and boating and instancing a number of cases where permission to fish resulted adversely. In most of these cases, however, the relation of water filtration was not stated.

In a still further discussion,‡ the following conclusions were offered by George A. Johnson, M. Am. Soc. C. E.:

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* Chicago, Ill.

† *Transactions*, Am. Soc. C. E., Vol. LIX (1907), p. 367.

‡ *Journal*, Am. Water Works Assoc., July, 1921.

(*a*).—Maintain the catchment area in as sanitary a condition as practicable; that is, guard against gross pollution entering the streams and lakes which drain the water-shed.

(*b*).—Store the water in natural lakes or artificial reservoirs, provided such storage is available or dictated by sound engineering principles.

(*c*).—Coagulate and filter.

(*d*).—Sterilize.

These comments, however, are not specific as to the use and protection of reservoir supplies.

Apparently, this matter is not finally settled in all cases, as indicated by a recent Act submitted to the Massachusetts Legislature relative to the use of Lake Cochituate in the Town of Natick for boating and fishing. The officers of the New England Water Works Association have protested against the passage of this bill.* Efforts should be directed to (1) the control of human activities to prevent undue pollution; and (2) the control of natural processes to prevent deterioration of physical qualities.

Principal Considerations.—The operation of reservoirs for water supply may be discussed from two principal considerations: (*a*) the use of reservoirs other than for water supply purposes; and (*b*) their maintenance for water supply purposes.

The first consideration includes the regulation of the reservoir for bathing, boating, fishing, picnicing, and the development of the shores into camps, resorts, and residential and industrial districts. The effect of these various uses on the condition of the water as affecting the safe load on water filtration plants is the present-day criterion.

The second consideration includes the necessary operations for the maintenance of the reservoir, such as the control of objectionable growth of microscopic organisms, the proper draining and clearing of the banks, the protection of the shores, the regulation of storage, the handling of silt, the control of fish life, the drainage of swamps, the control of erosion, and the like.

In connection with the approaching completion of the impounding reservoir at Decatur, Ill., the speaker, during the past several months, has gathered some data indicating practice along these general lines in a number of other situations. (See Tables 5 and 6.)

Summary of Data on the Use of Impounding Reservoirs.—Table 5 shows the practice with filtered water supplies. In general, it is indicated that no bathing is allowed, with the exception of Youngstown, Ohio, and at Fort Worth, Tex., some bathing is permitted. There is indicated a more general permission for boating, as this is forbidden entirely in only seven towns out of the fourteen records available. Fishing from the shores is almost universally permitted, and quite generally also from boats, although this latter practice is commonly regulated through permits or licenses. Picnicing and camping seems also to be allowed, with the establishment of camps and resorts permitted in about one-half the cases. Some method of patrol is almost always undertaken, and among the more comprehensive of these methods are daily inspections by five mounted patrolmen over a drainage area of 110 sq. miles for the Oakland, Cal., water supply.

* *Journal*, New England Water Works Assoc., September, 1921.

TABLE 5.—WATER SUPPLY DATA: USE OF IMPOUNDING RESERVOIRS FOR FILTERED WATER SUPPLIES.

City.	Reservoir Data:		Use of Reservoir:						Method of patroling.
	Capacity, in million gallons.	Area, in acres.	Bathing.	Boating.	Fishing from shores.	Fishing from boats.	Picnicing and camping.	Resorts.	
Akron, Ohio	2 400	769	No	No	Yes	Some	Some	No	One man
Columbus, Ohio	1 720	363	No	No	Yes	No	Yes	No	Two river patrols.
Dallas, Tex.*	....	....	No	Yes	Yes	Yes	Yes	No	
Fort Worth, Tex	2 145	....	Yes	Yes	Yes	Yes	....	....	Sanitary patrol.
Girard Water Company, Pa	....	....	No	No	Yes*	No	No	No	Caretaker at each reservoir.
Jacksonville, Ill	453	265	No	Yes	Yes	Yes	Yes	No	Under Park Board.
Mt. Vernon, Ill	350	....	No	Yes	Yes	Yes	Yes	Cottages	Some patroling.
Oklahoma, Okla	8 000	2 700	No	No	Yes	Yes	Yes	No	None.
Oakland, Cal	19 155	1 310	No	No	No	No	No	No	Five mounted patrols.
Ravenna, Ind	....	85	No	Yes	Yes	Yes	Yes	Cottages	Shores treacherous.
Reading, Pa	150	31	No	Some	Yes	Yes	Yes	Caretakers	Monthly.
Springfield, Mass	2 500	214	No	No	No	No	Little	No	Caretaker resident.
Wilkes-Barre, Pa	9 000	....	No	No	†	No	No	No	Daily patrol.
Youngstown, Ohio	9 000	1 700	Yes	Yes	Yes	Yes	Yes	Yes	None.

* Subject to general regulations.
† Limited fishing from shore permitted when water is filtered.

TABLE 6.—WATER SUPPLY DATA: USE OF IMPOUNDING RESERVOIRS FOR UNFILTERED WATER SUPPLIES.

City.	Reservoir Data:		Use of Reservoir:						Method of patroling
	Capacity, in million gallons.	Area, in acres.	Bathing.	Boating.	Fishing from shores.	Fishing from boats.	Picnicing and camping.	Resorts.	
Altoona, Pa	1 030	130	No	No	No	No	No	No	Daily patrol.
Bellevue, Ohio	250	....	No	No	Yes	No	No	No	Patrolmen, May to November.
Centralia, Ill	850	....	No	Yes	Yes	Yes	Yes	Yes	None.
New London, Conn	600	225	No	No	No	No	No	No	Regular trips.
Omaha, Nebr	100	....	No	No	No	No	No	No	Watchman.
Paris, Ill.*	300	100	Yes	Yes	Yes	Yes	Yes	Yes	None.
Portland, Ore	(?)	500	No	No	No	No	No	No	U. S Forest Service.
Rochester, N. Y	7 328	2 476	No	No	No	No	No	No	Constant inspection.
San Francisco, Cal	....	....	No	No	No	No	No	No	Seven mountain men.
Seattle, Wash	270	....	No	No	No	No	No	No	Fenced in.
Syracuse, N. Y	Skaneateles Lake		No	No	Yes	Yes	Yes	Cottages	Patroled by water and road.

* Water not used for drinking.

Table 6 indicates the data for unfiltered water supplies. In only one case is bathing reported as permissible, that is, at Paris, Ill., in which city the water is not used for drinking. Boating also is forbidden, whereas fishing from the shores is forbidden in seven cases out of eleven. Fishing from boats is also generally forbidden, and this is also true of picnicing, camping, and the development of resorts.

In the case of the impounded or lake supplies for Boston, Mass., and New York City, very careful control is exercised, fishing and some boating is allowed by permit, and some bathing occurs near some of the New York reservoirs.

Practice thus indicates a somewhat freer use of reservoir waters in the case of filtered supplies.

Comments on Bathing.—The matter of bathing appears to be the most important consideration. It should be considered not only for its effect on the quality of the water in the reservoir, but also for its effect on the bathers themselves. The effect of crowded bathing beaches on the bathers has recently called for the serious attention of the Health Departments in Chicago, Ill., Milwaukee, Wis., and elsewhere along the Great Lakes. A suggestion is offered that better bathing facilities can be provided in specially designed pools or ponds, in which the circulation and purification of the water can be controlled. With present-day understanding of the quite general distribution of typhoid carriers, the regulation of bathing assumes increasing importance. For a short time in 1921, several public bathing beaches in Chicago were closed.

Maintenance of Reservoirs.—No large impounding reservoir for water supply can be left to itself, and some more or less continuous maintenance is required. In the first place, developments on the drainage area above the reservoir should be convassed frequently so that gross pollution may be avoided. In the second place, the development of undesirable microscopic growths must be watched and controlled. Methods of control include a reasonable removal of the causes of such growths, the destruction of the growths by the application of minute quantities of copper sulphate, and the improvement of the water by aeration before filtration.

The third consideration involves the maintenance of satisfactory shore conditions, which can often be accomplished by draining, clearing, and burning over marginal areas during low water. It is believed to be particularly important that constant patroling should be undertaken, probably by boat and automobile, so that the establishment of gross nuisances may be prevented at all times, as well as minor infringements of the reservoir use.

Recent English Data.—The speaker's attention has recently been called to a moderate use of impounding reservoirs for fishing in England. A book recently published, entitled "Trout in Lakes and Reservoirs", by Ernest Phillips, offers some interesting data. It appears that within the last few years about forty or more English cities have stocked their city water supply reservoirs with fish and opened them to the public. In some of these cities, fishing tickets are issued, which cost at Manchester 25 cents per day and at Huddersfield 50 cents per fishing day. The speaker is not informed as to the extent of water filtration at these supplies, nor as to which, if any, are

chlorinated. It is the general understanding in the United States that few of them are filtered. An interesting account of these supplies appears in the *Outlook* for October 12th, 1921.

Rules and Regulations.—A number of rules and regulations governing the use of reservoirs have been published and others have been suggested. Among the most complete are those of the Massachusetts State Board of Health, the Department of Water Supply, Gas and Electricity of New York City, a suggested law by W. H. Dittoe, M. Am. Soc. C. E., Chief Engineer of the Ohio State Board of Health, and ordinances recently passed at Dallas, Tex., and elsewhere. The aim of these rules and regulations is to prevent entirely the use of the reservoir for fishing, bathing, boating, and the like, or to permit some boating and fishing under careful regulation through permits and licenses. Bathing appears to be quite generally denied.

Summary and Conclusions.—The brief time available for the presentation of this subject of the operation of reservoirs for water supplies, and the multitude of differing local conditions, do not permit general definite conclusions. Practice and experience indicate that bathing, except under most favorable conditions, should not be permitted, and that consideration should be given to the protection of bathers as well as to the protection of the water supply. With unfiltered supplies, the tendency is frequently against boating, fishing, and the like. In the case of filtered supplies, the use of reservoirs for such purposes appears to be increasing under the most thorough and careful sanitary patrol.

DISCUSSION ON WATER SUPPLY AND WATER PURIFICATION

By Messrs. Louis L. Tribus, G. F. Catlett, and John R. Baylis.

Louis L. Tribus,* M. Am. Soc. C. E.—For a great many years the speaker's firm had under its observation and care a Western water-works system, the supply for which came from a flashy river, varying greatly in alkalinity and turbidity.

During different seasons and storm conditions, the alkalinity ranges from 40 to 320 parts per million and the turbidity, from zero to 5 000 (5 000 marks the limit on the gauge); how much worse it has been is not known. The point of interest, however, was the treatment of the water with coagulant. Taking the average through the year, $1\frac{1}{2}$ to $2\frac{1}{2}$ grains of sulphate of alumina were required per gallon, and, at times, it was necessary to increase the dose greatly. On one occasion, when the turbidity was at the maximum, the water would not yield until the alum dose was increased to 20 grains. Then, it suddenly yielded and although the conditions seemed to be the same, the alum dose was decreased at once to 11 and then to 7 grains, with perfect results.

Another fact of special interest is that an advantage was gained by dosing the raw water at different stages in its passage. From the river, the water first passed through four successive sections of a sedimentation or coagulation basin; and then to mechanical gravity open filters. Alum was first added at the point where the centrifugal pump took the water for discharge into the first basin, and since the water entered near the bottom of that first basin, a second dose could be given. The flow passed through a port about 3 ft. from the bottom of the wall, between the first and second chamber, and, from the second to the third section, it passed at a little higher elevation. At this port a third dose of alum was added. The water then passed through the third basin into the fourth, and through that and a short pipe line to the filters. The final dose of alum was given just before the water reached the filters. It was found that by using these successive proportional charges, far better results could be obtained than by using the full dose at any one point. The clearest water was neither near the bottom nor at the top, but from about 6 in. to 1 ft. below the top.

G. F. Catlett,† Assoc. M. Am. Soc. C. E.—The speaker was much interested in the discussion on colored waters by Mr. Weston, particularly in his reference to such waters in Eastern North Carolina. Considerable study has been, and is being, given to the treatment of these waters as they occur in that State and some interesting results were published regarding the treatment of the colored water at Wilmington, N. C.

The outstanding facts brought out in connection with these studies, is that the treatment of colored water requires close and scientific regulation of chemicals and of the general treatment, and does not require as long a retention time in the coagulating basins and probably not as low a velocity as is required by turbid waters.

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* New York City.

† Raleigh, N. C.

In the case of the colored waters of North Carolina, and it is true in general of those found elsewhere, the alkalinity is rather low, and additional alkali must be applied. In applying this alkali a careful adjustment of the ratio of aluminum sulphate to alkali was found to be necessary, so that the hydrogen-ion concentration will be close to the iso-electric point. If this is done, coagulation will occur much more quickly than in the case of turbid, clay waters, and the "floc" will be in larger, heavier particles. If slightly more alkali is added than is needed to secure the iso-electric point, coagulation will occur in such small particles that no satisfactory sedimentation or filter efficiency can occur. It is obvious that this required control must be under complete laboratory test and under the supervision of an operator thoroughly conversant with the principles involved. In order to secure a factor of safety against free alum going over to the filters, it is found necessary to add a small quantity of alkali as a secondary treatment after coagulation and sedimentation. The quicker coagulation secured under proper treatment, and the larger and heavier "floc", are the reasons for suggesting a lower basin retention time and a higher permissible velocity in the basin.

In treating the colored water at Wilmington, where complete laboratory tests and scientific control is provided, satisfactory color removal is secured with economical use of chemicals. At Elizabeth City and Lumberton, where there is no trained technical supervision, rather unsatisfactory results are secured, with a much higher consumption of chemicals.

The reference to the brackish water at Wilmington, is entirely aside from the difficulty encountered in treating a colored water. Following an abnormal dry spell, such as has been experienced this year (1921), in conjunction with certain wind and other conditions in the ocean, sea water is backed up thirty miles distant to the intake. The October, 1921, report from the Wilmington plant shows chloride content as high as 1,600 parts per million. The same trouble is experienced to a greater or less extent at other towns along the coast. As the condition exists for a short period only, the remedy is raw-water storage sufficient to carry over the duration of salt water.

It is of interest that this condition at Wilmington, in its more aggravated form, was coincident with the construction of locks and dam on the river above the intake.

JOHN R. BAYLIS,* ASSOC. M. AM. SOC. C. E.—Hydrogen ions having been mentioned by Messrs. Whipple, Hazen, and Weston, the speaker will discuss that subject briefly. The laboratory at the Montebello Filters, of the Baltimore City Water Department, was one of the first to use hydrogen ion in plant control and it is now one of the most important tests. The water at Baltimore is comparatively soft, the alkalinity being about 40 and the total hardness about 50. It is corrosive before treatment, and after the application of alum, unless neutralized by an alkali, it is so corrosive that when it reaches the consumer it may look about as turbid as it did before filtration, due to corrosion of the pipes. It is necessary, therefore, to reduce its cor-

* Baltimore, Md.

rosive qualities, which is now done by adding lime near the outlet of the mixing basin.

Mixing basins are a feature frequently overlooked in filter design, and it may be well to discuss briefly this part of the plant which helps bring about the chemical reactions. Some years ago the speaker had the experience of trying to operate a plant which had no mixing basin, and after one year of unsatisfactory efforts, a basin was constructed, and immediately the desired results were obtained. Some interesting experiments have been made in California on mechanical agitation to produce the desired mixing, and the speaker believes there is something in that method, because in Baltimore, several years ago, when iron and lime were used as a coagulant, good coagulation was frequently hard to obtain. It required about $\frac{1}{2}$ hour for the water to pass through the mixing basin, which is longer than in most plants. Experiments showed that when this water, which had received the $\frac{1}{2}$-hour mix without producing a satisfactory coagulation, was stirred slowly for 20 or 25 min., excellent results were produced. In recent experiments on mechanical agitation, the speaker has found the time element important, as well as the saving in chemicals. A turbidity requiring about $\frac{3}{4}$ grain per gal. of alum and $\frac{1}{2}$ hour in the mixing basin of the plant, could be properly coagulated with $\frac{2}{5}$ or $\frac{1}{2}$ grains per gallon, if stirred for 25 or 30 min. with the experimental apparatus.

It has been stated that coagulation with colored waters is difficult, but the precipitation is very rapid when once obtained. The chances are that this colored water was overdosed with chemicals, which produces rapid precipitation after the floc is formed. A longer period of mixing may change the results with considerable saving in chemicals. The speaker favors large coagulating basins even though there may be rapid precipitation. If the total capacity is not needed, longer periods between cleaning may be allowed. From experiences in Baltimore, larger basins are preferred. Chemical reactions are sometimes slow, and, at times, it seems that the reactions are not complete until after the water has passed the filters. The capacity of the basins, however, does not always give an indication of the time in passing through, since float tests have shown that in less than 1 hour part of the water may pass through basins having capacities of $2\frac{1}{2}$ or 3 hours.

It is possible to prevent corrosion almost entirely by adjusting chemical treatment to the proper hydrogen-ion concentration. With soft waters this may mean a PH value of nearly 9.0 and with hard waters the value may be as low as 7.0, or even less, without producing corrosion. For most waters, in adjusting the hydrogen-ion concentration when alum is used, the alkali should be added after filtration. Preparations are now being made at Baltimore to add the lime after filtration. The water before treatment has a PH value of about 7.0, which is reduced to about 6.5, or less, by the addition of alum, and it is later adjusted to about 8.4 or 8.6 by the addition of lime. There seems to be nearly complete precipitation of the alum if the PH value is not below 6.5, but beyond that it may not be complete. On account of the low alkalinity in some waters, it may be necessary to add some of the alkali before the alum, but only enough should be added to produce a PH value of about 6.5 after application of the alum, and then adjust it to the proper

hydrogen-ion concentration after filtration. In Baltimore, it is possible to re-dissolve about one-half the aluminum hydrate by applying lime in doses slightly higher than are necessary to the coagulated water. These re-dissolved compounds may have no effect on the water, but chemicals are wasted by this practice.

The hydrogen-ion concentration is one of the easiest determinations to make, and possibly is the only test that will enable the filter operator to be sure of the corrosive qualities of the water treated. As soon as the proper concentration is determined for a certain water, the necessary adjustments can be made, and complaints about corrosion or red water will cease. The speaker believes that to-day the hydrogen-ion determination is one of the most important tests.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

This Society is not responsible for any statement made or opinion expressed in its publications.

TENTATIVE SPECIFICATIONS FOR STEEL RAILWAY BRIDGES

SUBMITTED AS A PROGRESS REPORT OF THE SPECIAL COMMITTEE ON SPECIFICATIONS FOR BRIDGE DESIGN AND CONSTRUCTION*

WITH DISCUSSION BY MESSRS. HENRY B. SEAMAN, F. E. TURNEAURE, AND BURTON R. LEFFLER.

COMMITTEE

HENRY B. SEAMAN, *Chairman*,

H. C. BAIRD
C. W. HUDSON
M. S. KETCHUM
B. R. LEFFLER
A. F. ROBINSON
F. E. TURNEAURE
J. R. WORCESTER

November 16th, 1921.

* To be presented to the Annual Meeting, January 18th, 1922.

TO THE PRESIDENT AND BOARD OF DIRECTION,
AMERICAN SOCIETY OF CIVIL ENGINEERS:

The Special Committee on Specifications for Bridge Design and Construction submits the following report of progress of the work accomplished to the present time.

The first meeting of the Committee was held on December 13th, 1920, for the purpose of organization, and to formulate methods of procedure in its work. Mr. Henry B. Seaman was elected Chairman, and Mr. H. C. Baird, Secretary, of the Committee. The Committee then resolved itself into seven Sub-Committees, to collect data and make recommendations in the various branches of its work. Each of these Sub-Committees was composed of the membership of the whole, with the following Sub-Chairmen appointed by the Chairman of the Committee:

Highway Loading and Impact	M. S. Ketchum
Railway Loading and Impact	F. E. Turneaure
Substructure and Foundations	J. R. Worcester
Allowable Unit Stresses	B. R. Leffler
Shop and Erection Methods	A. F. Robinson
Reinforced Concrete Structures	C. W. Hudson
Materials	J. E. Greiner

It was decided by the Committee that the most practicable order of procedure would be first to outline a general specification for fixed spans of steel railroad bridges, which could be modified to meet the requirements of steel highway bridges, including highway loading, and with the loading of these two classes of bridges as a basis, the reinforced concrete bridges could then be taken up for consideration.

The Committee has held five meetings, three of which were of two days' session, and a sixth meeting will be held this year. All meetings were well attended, and, at several, a full attendance was obtained.

The Committee, at various times, has considered the enlargement of its membership, and it was decided that the work could be managed most expeditiously without adding to its number for the present, while its work is purely tentative. It is the purpose of the Committee to submit tentative forms to the membership of the Society for full, exhaustive, and constructive discussion, thus forming the entire Profession into a "Committee of the Whole," so to speak, and obtaining the broadest possible views as to practice and future procedure. This discussion will not be confined to the membership of the Society, but will be welcomed from all sources having experience and judgment on the subject. It may then seem advisable either to enlarge the Committee or to bring in outside judgment in the formulation of the final specification.

In conformity with this program, the Committee submits herewith to the Society, in tentative form, for discussion, a General Specification for the Design and Construction of Steel Railroad Bridges.

Respectfully submitted,

COMMITTEE:
H. C. BAIRD,
C. W. HUDSON,
M. S. KETCHUM,
B. R. LEFFLER,
A. F. ROBINSON,
H. B. SEAMAN,
F. E. TURNEAURE,
J. R. WORCESTER.

For the Committee,
HENRY B. SEAMAN,
Chairman,
HOWARD C. BAIRD,
Secretary.

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TENTATIVE SPECIFICATIONS FOR STEEL RAILWAY BRIDGES

SECTION A.—INTRODUCTORY.

1.—These specifications cover the design of fixed span bridges.

2.—Definitions of Terms.—The term, "Engineer", refers to the purchaser's engineer or to his authorized representative. The term, "Inspector", refers to the inspector or inspectors acting under the authority of the Engineer. The term, "Contractor", refers to the manufacturing or fabricating contractor, party to the contract.

3.—Proposals.—Bidders shall submit proposals conforming to the terms in the letter of invitation. The proposals preferably shall be based on plans and specifications furnished by the Purchaser, showing the general dimensions necessary for designing the structure, the stresses, and the general or typical details. Invitations covering work to be designed or erected by the Contractor shall state the general conditions at the site, such as track spacing, character of foundations, old structures, traffic conditions, etc.

4.—Drawings to Govern.—Drawings shall govern in cases where they are not in agreement with the specifications.

5.—Patented Devices.—The Contractor shall protect the Purchaser against claims on account of patented devices or parts proposed by him.

SECTION B.—DRAWINGS.

6.—Approval of Drawings.—If shop drawings are not furnished by the Purchaser, the Contractor shall submit for approval before work is commenced duplicate prints of all shop drawings, which drawings shall be delivered to the Purchaser on completion of the contract. Any material ordered by the Contractor prior to the approval of the drawings shall be at his risk.

7.—Shop Drawings.—Shop drawings shall be made on the dull side of tracing cloth, 24 by 36 in., including the margin, which shall be 1½ in. at the left end and ½ in. wide at the other edges. The title shall be in the lower right-hand corner. No change shall be made on approved drawings without the written consent of the Engineer.

8.—Approval of shop drawings by the Engineer shall not relieve the Contractor from responsibility for shop dimensions, shop fits, or field connections.

SECTION C.—GENERAL FEATURES.

9.—Material.—Structures shall be made wholly of structural steel except where otherwise specified. Cast steel preferably shall be used for shoes and bearings. Cast iron may be used only where specifically authorized by the Engineer.

10.—Type.—The type of bridges to be used for various span lengths shall preferably be as follows:

Rolled beams up to....................	30 ft.
Plate girders from.....................	30 to 125 ft.
Riveted trusses from..................	100 ft. and over.
Pin-connected trusses from............	175 ft. and over.

11.—Number of Trusses.—For double-track, through bridges, two trusses shall be used. Four-track bridges may be constructed as two double-track bridges, side by side, or may have three trusses, as directed.

12.—Spacing of Trusses.—The width between center of trusses or girders shall be sufficient to prevent overturning by the specified lateral forces and in no case less than one-twentieth of the span.

13.—Clearance.—The clearance on straight track shall be not less than that shown in Fig. 1. On curves, additional provision shall be made for a car 80 ft. long, 14 ft. high on top of a 6-in. rail and 60 ft. between truck centers, with allowance for super-elevation of the outer rail. Unless specified otherwise, the super-elevation of the outer rail shall be ¾ in. for each degree of curvature with a maximum of 8 in.

14.—Skew Bridges.—In skew bridges without ballasted floors, the end stringers or end girders for each track shall be square with the track.

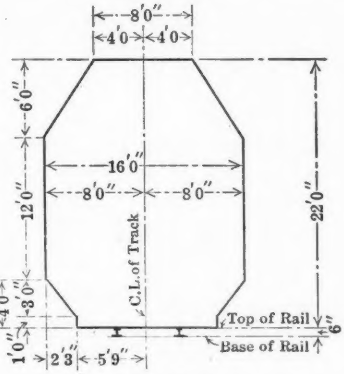


FIG. 1.

15.—Timber Floor.—Ties shall be not less than 10 ft. long, spaced not more than 6 in. apart, and dapped ½ in. over the lowest part of the girder or stringer. They shall be hook-bolted to girders or stringers as required by the Engineer and shall be secured against bunching by wooden guard-rails, 6 by 8 in., notched 1 in., and fastened to each tie by a ¾-in. bolt. Where subjected to bending, the stress on extreme fibers shall not exceed the unit stress for timber given in Article 201, assuming a maximum wheel load with 100% impact to be distributed equally over three ties.

16.—Ballasted Floors.—Ballasted floors shall have at least 6 in. of ballast under the ties, and the ballast shall be assumed as level with the base of rail, the weight of the ties being neglected. The live load, assumed as 15 000 lb. per lin. ft. of track, shall be considered as uniformly distributed over a width of 10 ft.

17.—Dimensions for Calculations.—For calculation of stresses the length shall be the distance between centers of bearings for trusses and girders, between centers of trusses for floor-beams, and between centers of floor-beams for stringers. The depth shall be the distance between centers of pins for pin-connected trusses, between centers of gravity of chords for riveted trusses, and between centers of gravity of flanges for plate girders, unless net section modulus is used.

18.—Spans with floor system shall preferably have end floor-beams.

SECTION 1.—LOADS AND STRESSES.

101.—Weight of Materials.—In estimating the weight of the structure, for the purpose of computing the stresses therein, the following unit weights shall be used:

Steel	490 lb. per cu. ft.
Concrete	150 " " " "
Sand and gravel	100 " " " "
Asphalt-mastic	150 " " " "
Bituminous macadam	130 " " " "
Granite	170 " " " "
Paving bricks	150 " " " "
Spruce, white and red pine, and Douglas fir.	3 " per ft. B. M.
Southern long-leaf pine	4 " " " "
Oak and birch	5 " " " "
Creosoted timber	5 " " " "

The rails and fastenings shall be assumed to weigh 150 lb. per lin. ft. for each track.

102.—Loads.—Stresses shall be shown separately for the following: Dead load, live load, impact, centrifugal force, lateral and longitudinal forces,

and the members shall be proportioned for the combination giving maximum results, except as otherwise provided.

103.—Dead Load.—The dead load shall consist of the entire weight of the superstructure, estimated in accordance with unit weights given in Article 101.

104.—Live Loads.—The live load for each track shall be as shown in Fig. 2 or Fig. 3, and that producing the greater stress shall be used.

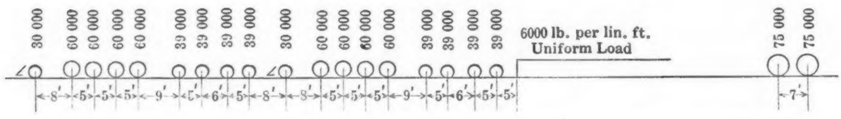


FIG. 2. FIG. 3.

For special cases the live load may be varied as required by the Engineer.

105.—Multiple Tracks.—For multiple-track bridges, the live load with impact shall be reduced as follows:

For two tracks loaded..............	10 per cent.
" three " "	20 per cent.
" four " "	30 per cent.

106.—Impact.—The impact or dynamic increase in railway live load shall be determined by the following formula:

$$I = S \frac{300}{300 + \frac{L^2}{100}}$$

in which

I = impact or dynamic increment to be added to live-load stress;
S = computed maximum live-load stress;
L = span length for chords and end posts and loaded length for web members, neglecting the load in advance of the panel point considered.

107.—For bridges designed exclusively for electric traction, the impact shall be taken as one-half that given by the above formula. Impact shall not be added to stress produced by longitudinal or lateral forces.

108.—Overload.—All members and details shall be proportioned so that an increase of 50% in live load with impact will not increase the unit stress given in the specifications more than 50 per cent.

109.—Lateral Forces.—The lateral forces shall consist of a horizontal moving load equal to 30 lb. per sq. ft. on one and one-half times the vertical projection of the structure on a plane parallel with its axis (but never less than 200 lb. per lin. ft. at the loaded chord, and 150 lb. per lin. ft. at the unloaded chord), and a moving load of 10% of the specified live load on one track applied 8 ft. above the base of rail.

110.—If a moving load of 50 lb. per sq. ft. on one and one-half times the vertical projection of the unloaded structure on a plane parallel with its axis, produces greater stresses than the lateral force defined in Article 109, it shall be provided for.

111.—In calculating the stresses in viaduct towers due to lateral force, the viaduct shall be considered as loaded on either one or both tracks, with empty cars weighing 1 200 lb. per lin. ft., with a wind pressure of 300 lb. per lin. ft. applied 8 ft. above the base of rail.

112.—Centrifugal Force.—Where structures are on curves, the centrifugal force assumed as acting 6 ft above the base of rail shall be computed by the formula:

$$C = \frac{0.067\, W\, V^2}{R}$$

in which,

C = horizontal centrifugal force;
R = radius of curve, in feet;
W = live load plus impact;
V = speed, in miles per hour.

113.—Longitudinal Force.—Provision shall be made in the design for the effect of a longitudinal force of 20% of the live load on one track only, applied 6 ft. above the top of rail. In structures (such as ballasted deck bridges of only three or four spans) where, by reason of continuity of members or frictional resistance, the longitudinal force will be largely directed to the abutments, its effect on the superstructure shall be taken as one-half that specified above.

114.—Temperature Stresses.—Allowance shall be made for variation in temperature of 120° Fahr., by providing members of sufficient strength to resist the temperature stresses.

Section 2.—Unit Stresses.

201.—The several parts of the structure shall be proportioned so that the unit stresses will not exceed the following in structural grade and rivet steel:

	Pounds per square inch.
Axial tension, net section	16 000
Axial compression,* gross section (*a*)	$15\,000 - 50\,\frac{l}{r}$
but not to exceed	12 500
Axial compression,* gross section (*b*)	$12\,500 - 0.25\left(\frac{l}{r}\right)^2$
Axial compression,* gross section (*c*)	$\dfrac{16\,000}{1 + \dfrac{l^2}{13\,500\, r^2}}$
but not to exceed	14 300

in which,

l = the length of the member, in inches;
r = the least radius of gyration of the member, in inches;

	Pounds per square inch.
Tension in extreme fibers of rolled shapes, built sections and girders, net section	16 000
Tension in extreme fibers of pins	24 000
Shear in plate-girder and I-beam webs, net section	10 000
Shear in power-driven rivets and pins	12 000
Bearing on power-driven rivets, pins, outstanding legs of stiffener angles, and other steel parts in contact	24 000

* These column formulas are in common use by railroad engineers, and each one gives results consistent with the other unit stresses herein recommended when used within the limits hereinafter specified. The Committee is unable at this time to agree on any one to the exclusion of the other.

The above mentioned values of shear and bearing shall be reduced 25% for countersunk rivets and turned bolts, and 15% for field rivets.

Bearing on expansion rollers, per linear inch................ 600 d

d = the diameter of the rollers, in inches.

For cast steel in shoes and pedestals the above-mentioned unit stresses for rolled steel shall apply.

Tension in extreme fibers of:	Pounds per square inch.
White oak, Douglas fir, and Southern long-leaf pine.........	2 000
White and red pine and spruce............................	1 200
Bearing on granite masonry...............................	800
Bearing on sandstone and limestone masonry................	400
Bearing on concrete masonry, 1:2:4 mix...................	600

202.—The diagonal tension in webs of girders and rolled beams at sections where maximum shear and bending occur simultaneously, shall not exceed 16 000 lb. per sq. in.

203.—Effective Bearing Area.—The effective bearing area of a pin, bolt, or rivet shall be its diameter multiplied by the thickness of the piece, except that for countersunk rivets half the depth of the countersink shall be omitted.

204.—Effective Diameter of Rivets.—In proportioning rivets, the nominal diameter shall be used.

205.—Reversal of Stress.—Members subject to reversal of stress under the passage of the live load shall be proportioned as follows: Determine the resultant tensile stress and the resultant compressive stress and increase each by 50% of the smaller; then proportion the member so that it will be capable of resisting either increased resultant stress. The connections shall be proportioned for the sum of the resultant stresses.

206.—Combined Stresses.—Members subject to both axial and bending stresses (including bending due to floor-beam deflection) shall be proportioned so that the combined fiber stresses will not exceed 16 000 lb. per sq. in. In members continuous over panel points, three-fourths of the bending stress computed as for simple beams shall be added to the axial stress.

207.—When secondary stresses are not considered, members subject to stresses produced by a combination of dead and live loads, impact and centrifugal force, with lateral or longitudinal forces, or bending due to lateral action, may be proportioned for unit stresses 25% greater than those specified in Article 201. When secondary stresses are included, the unit stresses may be increased $33\frac{1}{3}$ per cent. In no case shall the section be less than that required for dead and live loads, impact and centrifugal force at the unit stresses specified in Article 201, or less than that required if secondary stresses are not considered.

208.—Secondary Stresses.—Secondary stresses shall be avoided where possible in designing and detailing. In ordinary trusses without sub-paneling, no account usually need be taken of the secondary stresses in any member whose width measured in the plane of the truss is less than one-tenth of its length. Where this ratio is exceeded, or where sub-paneling is used, secondary stresses due to deflection of the truss shall be computed.

209.—Compression Flanges.—The gross area of compression flanges of plate girders or I-beams shall not be less than the gross area of the tension flanges, but the stress per square inch shall not exceed $16\,000 - 150\,\frac{l}{b}$, in which,

l = the length of the unsupported flange, between lateral connections or knee-braces; and

b = the flange width.

SECTION 3.—DETAILS OF DESIGN.

301.—Parts Accessible.—Details shall be designed so that all parts will be accessible for inspection, cleaning, and painting. Closed sections shall be avoided wherever possible. Pockets or depressions which would hold water shall have efficient drain holes, or shall be filled with concrete.

302.—Limiting Lengths of Members.—The ratio of length to least radius of gyration shall not exceed 100 for main compression members, nor 120 for wind and sway bracing.

303.—The lengths of riveted tension members shall not exceed 200 times their least radius of gyration.

304.—Depth Ratios.—The depth of trusses preferably shall be not less than one-tenth of the span. The depth of plate girders preferably shall be not less than one-twelfth of the span. The depth of rolled beams used as girders and the depth of solid floors preferably shall be not less than one-fifteenth of the span. If less depths than these are used, the section must be increased so that the maximum deflection will not be greater than if these limiting ratios had not been exceeded.

305.—Eccentric Connections.—Where possible, members shall be connected so that their gravity axes will intersect at a point.

306.—Effective Area of Angles.—The effective area of single angles in tension shall be assumed as the net area of the connected leg plus 50% of the area of the unconnected leg. Single angles connected by lug-angles shall be considered as connected by one leg.

307.—Counters.—If web members are subject to reversal of stress their end connections preferably shall be riveted. Adjustable counters shall have open turnbuckles.

308.—Strength of Connections.—Connections shall have a strength at least equal to that of the members connected, regardless of the computed stress, and shall be made, as nearly as practicable, symmetrical with the axis of the member.

309.—Limiting Thickness of Metal.—Metal shall be not less than $\frac{3}{8}$ in. thick, except for fillers. Metal subject to marked corrosive influence shall be increased in thickness or protected against such influences.

310.—Pitch of Rivets.—The minimum distance between centers of rivet holes shall be three diameters of the rivet, but the distance preferably shall be not less than $3\frac{1}{2}$ in. for 1-in. rivets, 3 in. for $\frac{7}{8}$-in. rivets, and $2\frac{1}{2}$ in. for $\frac{3}{4}$-in. rivets. The maximum pitch in the line of stress for members composed of plates and shapes shall be 7 in. for 1-in. rivets, 6 in. for $\frac{7}{8}$-in. rivets, and 5 in. for $\frac{3}{4}$-in. rivets. For angles with two gauge lines and rivets staggered, the maximum pitch in each line shall be twice the amounts given above. If two or more web-plates are used in contact, stitch rivets shall be provided to make them act in unison. In compression members, the stitch rivets shall be spaced not more than 24 times the thickness of the thinnest plate in the direction perpendicular to the line of stress, and not more than 12 times the thickness of the thinnest plate in the line of stress. In tension members, the stitch rivets shall be not more than 24 times the thickness of the thinnest plate in either direction. In tension members composed of two angles in contact, a pitch of 12 in. may be used for riveting the angles together.

311.—Edge Distance.—The minimum distance from the center of any rivet to a sheared edge shall be $1\frac{3}{4}$ times the diameter of the rivet, and to a rolled edge $1\frac{1}{2}$ times the diameter of the rivet. The maximum distance from any edge shall be 8 times the thickness of the plate or angle-leg with a limit of 6 in.

312.—Size of Rivets in Angles.—The diameter of the rivets in any angle the size of which is determined by calculated stress shall not exceed one-fourth of the width of the leg in which they are driven. In angles the size of which is not so determined 1-in. rivets may be used in $3\frac{1}{2}$-in. legs, $\frac{7}{8}$-in. rivets in 3-in. legs, and $\frac{3}{4}$-in. rivets in $2\frac{1}{2}$-in. legs.

313.—Long Rivets.—Rivets which carry calculated stress and the grip of which exceeds four and one-half diameters shall be increased in number at least 1% for each additional $\frac{1}{16}$ in. of grip. If the grip exceeds six times the diameter of the rivet, specially designed rivets shall be used.

314.—Pitch of Rivets at Ends of Members.—The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivet for a distance equal to one and one-half times the maximum width of the member.

315.—Compression Members.—In built compression members, the metal shall be concentrated in the webs and flanges. Large columns and compression members of trusses shall have their segments connected by solid webs, if practicable. The thickness of each web shall not be less than one-thirtieth of the distance between the lines of rivets connecting it to the flanges. The thickness of cover plates shall not be less than one-fortieth of the distance between the nearest rivet lines.

316.—Outstanding Legs of Angles.—The width of outstanding legs of angles in compression (except when reinforced by plates) shall not exceed the following:

a.—For stringer flange angles, twelve times the thickness.
b.—For main members carrying axial stress, twelve times the thickness.
c.—For bracing and secondary members, sixteen times the thickness.

317.—Stay-Plates.—The open sides of compression members shall be provided with lacing-bars and shall have stay-plates as near each end as practicable. Stay-plates shall be provided at intermediate points where lacing is interrupted. In main members, the length of stay-plates shall be not less than one and one-quarter times the distance between the lines of rivets connecting them to the outer flanges, and the length of intermediate stay-plates shall be not less than three-quarters of that distance. Their thickness shall be not less than one-fiftieth of the same distance.

318.—Built Tension Members.—Tension members composed of shapes shall have their separate segments stayed together.

319.—Lacing.—The lacing of compression members axially loaded shall be proportioned to resist a shearing stress of $2\frac{1}{2}$% of the direct stress. The minimum width of lacing-bars shall be 3 in. for 1-in. rivets, $2\frac{3}{4}$ in. for $\frac{7}{8}$-in. rivets, $2\frac{1}{2}$ in. for $\frac{3}{4}$-in. rivets, and 2 in. for $\frac{5}{8}$-in. rivets. The thickness shall be made as required by Article 201, in which l shall be taken as the distance between connections to the main sections.

320.—In members composed of side segments and a cover plate, with the open side laced, one-half of the shear shall be considered as taken by the lacing. Where double lacing is used, the shear in the plane of the lacing shall be distributed equally between the two systems.

321.—Lacing-bars of compression members shall be spaced so that the $\frac{l}{r}$ of the portion of the flange included between their connections will be not greater than 40, and not greater than two-thirds of the $\frac{l}{r}$ of the member.

322.—In connecting lacing-bars to flanges, $\frac{5}{8}$-in. rivets shall be used for flanges less than $2\frac{1}{2}$ in. wide, $\frac{3}{4}$-in. rivets for flanges from $2\frac{1}{2}$ to $3\frac{1}{2}$ in. wide, and $\frac{7}{8}$-in. rivets for flanges $3\frac{1}{2}$ in., wide or more. Lacing-bars with at least two rivets in each end shall be used for flanges more than 5 in. wide.

323.—The angle of lacing-bars with the axis of the member shall be not less than 45° for double lacing and 60° for single lacing. If the distance between rivet lines in the flanges is more than 15 in. and a single rivet-bar is used, the lacing shall be double and riveted at the intersections.

324.—Splices.—Abutting joints in compression members faced for bearing shall be spliced on four sides. The gross area of the splice material shall be not less than 50% of the gross area of the smaller member.

325.—Joints for riveted work not faced for bearing, whether in tension or compression, shall be fully spliced.

326.—*Net Sections at Pins.*—In pin-connected riveted tension members, the net section across the pin-hole shall be not less than 140% and the net section back of the pin-hole, not less than 100% of the net section of the body of the member.

327.—*Net Section Defined.*—The net section of riveted members shall be the least area which can be obtained by deducting from the gross sectional area, the area of holes cut by any plane perpendicular to the axis of the member and parts of the areas of other holes on one side of the plane within a distance of 4 in., which are on a gauge line 1 in. or more from those of the holes cut by the plane, the parts being determined by the formula:

$$A\left(1 - \frac{P}{4}\right),$$

in which,

A = the area of the hole;
P = the distance, in inches, of the center of the hole from the plane.

328.—In determining the net section, the area of the rivet hole shall be taken $\frac{1}{8}$ in. larger than the nominal diameter of the rivet.

329.—*Pin-Plates.*—Where necessary to give the required section or bearing area, pin-holes shall be reinforced on each segment by plates, one of which on each side must be as wide as the outstanding flanges will permit. These plates shall be connected so as to transmit and distribute the bearing pressure uniformly over the full cross-section and to reduce the eccentricity of the segment to a minimum.

330.—*Indirect Splices.*—If splice-plates are not in direct contact with the parts which they connect, rivets shall be used on each side of the joint in excess of the number required in the case of direct contact to the extent of two extra lines for each intervening plate.

331.—Where rivets carrying stress pass through fillers, the fillers if over $\frac{5}{16}$ in. in thickness shall be extended beyond the connected member and the extension secured by sufficient rivets.

332.—*Forked Ends.*—Forked ends on compression members will be permitted only where unavoidable, in which case a sufficient number of pin-plates shall be provided to make the jaws of twice the sectional area of the member, and they shall be extended as far as necessary in order to carry the stress of the main member into the jaws.

333.—*Pins.*—Pins shall be long enough to secure a full bearing of all parts connected on the turned body of the pin. They shall be secured by chambered nuts or solid nuts with washers. Where the pins are bored, "through" rods with cap-washers may be used. The screw ends shall be long enough to admit of burring the threads. Members shall be secured against lateral movement on the pins.

334.—*Bolts.*—Where members are connected by bolts, the turned bodies of the bolts shall be long enough to extend through the metal. A washer at least $\frac{1}{4}$ in. thick shall be used under the nut. Bolts shall not be used except by special permission.

335.—*Upset Ends.*—Bars with screw ends shall be upset so that the area of the root of the thread will be at least 15% larger than in the body of the bar.

336.—*Expansion.*—Provision shall be made for expansion and contraction at the rate of 1 in. for every 80 ft. in length. In spans more than 250 ft. in length, provision shall be made for expansion in the floor.

337.—*Expansion Bearings.*—Spans 70 ft. and more in length shall have hinged shoes at both ends and rollers at one end; spans of less length shall be arranged to slide on smooth surfaces. Expansion ends shall be secured against lateral movement.

338.—Fixed Bearings.—Bearings at ends of spans shall be firmly secured in position.

339.—Expansion Rollers.—Expansion rollers shall be not less than 6 in. in diameter; they shall be connected by substantial side-bars; and they shall be effectually guided so as to prevent lateral movement, skewing, or creeping.

340.—Pedestals and Shoes.—Pedestals and shoes preferably shall be made of cast steel. The difference between the top and bottom bearing widths shall not exceed twice the depth. For hinged bearings, the depth shall be measured from the center of the pin. Where built pedestals and shoes are used, the web-plates and the angles connecting them to the base-plates shall not be less than $\frac{3}{4}$ in. thick. If the size of the pedestal permits, the webs shall be rigidly connected transversely. The minimum thickness of the metal in cast-steel pedestals shall be 1 in. Pedestals and shoes shall be constructed so that the load will be distributed uniformly over the entire bearing.

341.—Inclined Bearings.—For spans on an inclined grade and without hinged bearings, the sole plates shall be beveled so that the sliding surface will be level.

342.—Name Plates.—There shall be a name plate, showing in raised letters and figures the name of the manufacturer, the year of construction, and such other information as required, bolted to the bridge near each end at a point convenient for inspection.

Section 4.—Floors.

401.—Types of Floors.—Floors shall be of the "solid floor" type or shall consist of steel floor-beams and stringers with timber cross-ties supporting the rails as directed and shall be designed with special reference to stiffness.

403.—Spacing of Stringers.—In open floors there shall usually be two stringers under each track spaced 6 ft. 6 in., center to center. If four stringers are used under one track, each pair shall be spaced symmetrically about the rail.

404.—I-Beam Girders.—Rolled beams supporting timber decks shall be arranged with not more than four, and preferably not less than two, beams under each rail. The beams in each group shall be placed symmetrically about the rail and shall be spaced so as to permit cleaning and painting. They shall be connected by solid web diaphragms near the ends and at intermediate points, spaced not more than twelve times the flange width. Bearing-plates shall be continuous under each group of beams. Stiffeners shall be used where required.

405.—Floor Connections.—Floor-beams preferably shall be square to the girders or trusses and riveted directly to the girders or to the posts of through and deck truss spans. Stringers in through spans shall be riveted between the floor-beams and shall have connection angles not less than 4 in. in width, and not less than $\frac{5}{8}$ in. in thickness before facing. Shelf angles shall be provided on the floor-beams to support the stringers during erection, but the connection angles shall be sufficient to carry the whole load.

406.—Cross-Frames.—Where two lines of stringers are used under each track in panels more than 20 ft. in length, they shall be connected by cross-frames.

407.—Solid Floor Connections.—Where solid floors are connected to girders or trusses, the connection angles shall be not less than $\frac{5}{8}$ in. thick if they are to be faced, or $\frac{1}{2}$ in. thick if they are not to be faced.

408.—Proportioning Solid Floors.—Solid floors shall be proportioned by the section modulus of the net section.

Section 5.—Bracing.

501.—Design of Bracing.—All bracing shall be composed of shapes with riveted connections designed so as to avoid excessive bending stress in truss members.

502.—Lateral Bracing.—Top lateral bracing shall be provided in all through spans having sufficient head-room, in all deck spans except plate-girder spans carrying solid floors, and in all I-beam spans.

503.—Bottom lateral bracing shall be provided in all bridges except through spans where the solid floor is carried directly by the girders or trusses and deck-plate girder spans less than 50 ft. long.

504.—Portal and Sway Bracing.—Deck truss spans shall have bracing in the planes of the end posts of the truss. Through spans shall have sway bracing at each intermediate panel point of the top chord as deep as the head-room will allow, except that transverse struts of the same depth as the chords with knee-braces may be substituted for bracing less than 6 ft. in depth. They shall also have portal bracing, with knee-braces, as deep as the specified clearance will allow.

505.—Cross-Frames.—Deck plate-girder spans shall be provided with cross-frames at each end proportioned to resist all lateral forces, and shall have intermediate cross-frames at intervals not exceeding 18 ft.

506.—Laterals.—The smallest angle to be used in lateral bracing shall be $3\frac{1}{2}$ by 3 by $\frac{3}{8}$ in., located so as to clear the ties. There shall be not less than three rivets at each end connection of the angles. Angles shall be connected at their intersection by plates.

SECTION 6.—PLATE GIRDERS.

601.—Spacing of Girders.—The girders of deck bridges usually shall be spaced 6 ft. 6 in. between centers, except that:

a.—In single-track deck spans 75 ft. or more in length, the girder shall be spaced in accordance with Article 12, but not less than 7 ft. 6 in. between centers.

b.—In bridges on curves, the girders shall be spaced to conform to track requirements.

602.—Design of Plate Girders.—Plate girders shall be proportioned either by their net section modulus or by assuming that the flanges are concentrated at their centers of gravity. In the latter case, one-eighth of the gross section of the web, if properly spliced, may be used as flange section. For unusual sections, the net section modulus shall be used.

603.—Flange Sections.—The flange angles shall form as large a part of the area of the flange as practicable. Side-plates shall not be used except when flange angles exceeding 1 in. in thickness otherwise would be required.

604.—Flange plates shall be equal in thickness, or shall diminish in thickness from the flange angles outward. No plate shall have a thickness greater than that of the flange angles.

605.—Where flange cover-plates are used, one cover-plate of the top flange shall extend the full length of the girder. Other flange plates shall extend beyond the theoretical end a distance sufficient to develop the plate.

606.—Thickness of Web-Plates.—The thickness of web-plates shall be not less than $\frac{1}{20}\sqrt{D}$, where D represents the distance between flanges, in inches.

607.—Flange Rivets.—The flanges shall be connected to the web with a sufficient number of rivets to transfer to the flange section the horizontal shear at any point combined with any load that is applied directly on the flange. One wheel load, where ties rest on the flange, shall be assumed to be distributed over 3 ft.

608.—Flange Splices.—Splices in flange members shall not be used except by special permission of the Engineer.

609.—Web-Splices.—Splices in web-plates shall be equal in strength to the web in shear and moment. They shall extend the full depth between flanges and shall have at least two rows of rivets on each side of the joint.

610.—Stiffeners.—Stiffener angles shall be placed at end bearings and at points of concentrated load and shall be milled at bearing ends. Such stiffeners shall not be crimped and shall have outstanding legs proportioned for bearing and extending as nearly as practicable to the edge of the flange angles. All stiffeners in deck plate-girders shall be milled to bear on top flange angles.

611.—Intermediate Stiffeners.—Webs shall be stiffened by angles riveted thereto in pairs with outstanding legs not exceeding sixteen times their thickness and not less than 2 in. plus $\frac{1}{30}$ the depth of the girder. Intermediate stiffeners shall be placed at intervals not exceeding:

(*a*).—6 ft.;
(*b*).—The depth of the web;
(*c*).—The distance given by the formula, $d = \frac{t}{40}(12\,000 - S)$.

d = the distance between rivet lines of stiffeners, in inches;
t = the thickness of the web, in inches;
S = web shear, in pounds per square inch, at the point considered.

612.—If the depth of the web between the flange angles or side-plates is less than 50 times the thickness of the web, intermediate stiffeners may be omitted.

613.—Gusset-Plates in Through Spans.—In through spans, the top flanges shall be braced by means of gusset-plates or knee-braces, with webs connected to the floor-beams and to a stiffener angle on the girder, and extending usually to the clearance line. If the unsupported length of the inclined edge of the gusset-plate exceeds 18 in., the edge shall be stiffened by angles. The plate shall preferably form no part of the floor-beam web.

614.—In through spans with solid floors, there shall be knee-braces with $\frac{3}{8}$-in. webs extending usually to the clearance line at intervals of about 12 ft. Each knee-brace shall be riveted to the floor and to the girder and shall be properly stiffened.

615.—Ends of Through Girders.—If through plate girders project 2 ft. or more above the base of the rail, the upper corners at the ends of the bridge shall be protected by rounded or sloping brackets securely riveted to the girder, or the corners of the girders shall be rounded.

616.—Bearings on masonry preferably shall be raised above the masonry by metal pedestals.

617.—Sole plates shall be not less than $\frac{3}{4}$ in. thick, nor longer than 18 in.

618.—Anchor-Bolts.—Anchor-bolts shall be not less than $1\frac{1}{4}$ in. in diameter and shall extend 12 in. into the masonry and have washers under the nuts.

Section 7.—Trusses.

701.—Type of Truss and Members.—Trusses shall preferably have single intersection web systems, and for through spans, inclined end posts. The top chords and end posts shall be made of two or more segments with one cover-plate or with a web and shall have stay-plates and lacing on the open sides; stay-plates and lacing may be used in place of the cover in light sections. The bottom chords shall be of eye-bars or of riveted members of symmetrical shape. Web members shall be of symmetrical shape.

702.—In pin-connected trusses, riveted members shall be used for hip verticals and members performing similar function, and for the two panels at each end of the bottom chords in single-track spans.

703.—Camber.—The length of truss members shall be such that the camber will be equal to the deflection produced by dead load.

704.—Eye-Bars.—The thickness of the eye-bar shall be not less than one-eighth of the width, not less than 1 in., and not greater than 2 in. The diameter of the pin shall be not less than three-quarters of the width of the widest bar attached. Eye-bars when tested to destruction shall break in the body of the bar.

705.—Packing.—Eye-bars shall be packed as closely as practicable, but arranged so that adjacent bars in the same panel will not be in contact and so that no bar will be inclined to the plane of the truss more than $\frac{1}{16}$ in. per ft. They shall be secured against lateral movement, and bars of a set shall be placed symmetrically about the plane of the truss.

706.—Gusset-Plates.—In riveted trusses gusset-plates connecting the truss members shall be proportionate to the stresses, but shall not be less than $\frac{1}{2}$ in.

707.—Lifting Ends.—Provision shall be made for lifting the span at the ends.

708.—Masonry Plates.—Masonry plates shall be not less than 1 in. thick.

Section 8.—Viaducts.

801.—Bents and Towers.—Viaduct bents shall preferably be composed of two columns with a transverse batter of 1 to 6 for single-track and 1 to 8 for double-track structures, unless local conditions call for vertical columns. They shall usually be united in pairs to form towers, but where single bents occur the columns shall have hinged ends or shall be proportioned for bending from longitudinal forces.

802.—Bracing.—Transverse and longitudinal bracing shall be of shapes with riveted connections. Bottom struts shall be proportioned to resist stresses produced by temperature changes or shall be capable of moving the tower pedestals under the effects of temperature changes. Girders in tower spans shall be fastened at each end to the caps of the columns or to the transverse girder.

803.—Bases.—Column bases shall be arranged to slide on smooth surfaces where required.

804.—Depth and Spacing of Girders.—The depths of girders in viaducts shall preferably be uniform, and the girders shall generally be spaced uniformly throughout. In double-track structures the girders shall usually be spaced 6 ft. 6 in. for each track, with the outer girders resting on the column caps and the inner girders carried by transverse girders.

805.—Sole and Masonry Plates.—Sole and masonry plates shall be not less than $\frac{3}{4}$ in. thick.

806.—Anchorage.—Anchor-bolts for towers shall engage masonry weighing at least one and one-half times the uplift.

Section 9.—Materials.

A.—Structural Grade and Rivet Steel.

These specifications conform with the American Railway Engineering Association Standards of 1920 which, in turn, conform with those of the American Society for Testing Materials, Serial A7-16, except as to requirements for yield point, speed of testing machine, character of fracture, and surface defects. The paragraphs are herein titled and numbered to conform to the requirements of the general specifications of the Committee.

901.—Process.—Structural grade and rivet steel shall be made by the open-hearth process.

902.—Properties.—Test specimens of structural grade and rivet steel shall conform to the following requirements as to chemical and physical properties (except as modified in Articles 905 and 908):

	Structural steel.	Rivet steel.
Phosphorus, maximum		
Acid	0.06%	0.04%
Basic	0.04%	0.04%
Sulphur, maximum....................	0.05%	0.045%
Tensile strength, in pounds per square inch.	55 000-65 000	46 000-56 000
Yield point, in pounds per square inch, minimum	30 000	25 000
Elongation in 8 in., minimum percentage..	$\frac{1\,500\,000}{\text{Tensile Strength}}$	$\frac{1\,500\,000}{\text{Tensile Strength}}$
Elongation in 2 in., " " ..	22	....

903.—Ladle Analyses.—An analysis of each melt of steel shall be made by the manufacturer to determine the percentages of carbon, manganese, phosphorus, and sulphur. This analysis shall be made from a test ingot taken during the pouring of the melt. The chemical composition thus derived shall be reported to the Engineer.

904.—Check Analyses.—Analyses may be made by the Engineer from finished material representing each melt. The phosphorus and sulphur content thus determined shall not exceed that specified in Article 902 by more than 25 per cent.

905.—Eye-Bar Material.—In order to meet the minimum tensile strength of full-size annealed eye-bars, the Contractor may determine the tensile strength to be obtained in specimen tests, the range not to exceed 14 000 lb. per sq. in. and the maximum not to exceed 74 000 lb. per sq. in. The material shall conform to the requirements as to physical properties other than that of tensile strength specified in Articles 902, 908, and 910. Full-size tests of annealed eye-bars shall show a yield point of not less than 29 000 lb. per sq. in., an ultimate strength of not less than 54 000 lb. per sq. in., and an elongation of not less than 10% in a length of 20 ft. measured in the body of the bar. The fracture shall show a silky or fine granular structure throughout.

906.—Yield Point.—The yield point shall be determined by the drop of the beam of the testing machine.

907.—Speed of Testing Machine.—The cross-head speed of the testing machine shall be such that the beam of the machine can be kept balanced, but in no case shall the values given in the following table be exceeded.

Gauge length of specimen, in inches.	Maximum Cross-head Speed (Inches per Minute) in Determining:	
	Yield point.	Tensile strength.
2	0.5	2.0
8	2.0	6.0

908.—Modification in Elongation.—For structural steel more than $\frac{3}{4}$ in. in thickness, a deduction of 1 from the percentage of elongation in 8 in. specified in Article 902 shall be made for each increase of $\frac{1}{8}$ in. in thickness above $\frac{3}{4}$ in. to a minimum of 18 per cent. For structural steel less than $\frac{5}{16}$ in. in thickness, a deduction of 2.5 from the percentage of elongation in 8 in. specified in Article 902 shall be made for each decrease of $\frac{1}{16}$ in. in thickness below $\frac{5}{16}$ in.

909.—Bend Tests.—The test specimens for plates, shapes, and bars (except as specified in Articles 910, 911, and 912), shall bend cold through 180° without cracking, as follows:

(*a*).—For material $\frac{3}{4}$ in. or less in thickness, flat on itself.

(*b*).—For material more than $\frac{3}{4}$ in. to and including $1\frac{1}{4}$ in. in thickness, around a pin the diameter of which is equal to the thickness of the specimen.

(*c*).—For material more than 1¼ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

910.—The test specimens for eye-bar flats shall bend cold through 180° without cracking, as follows:

(*a*).—For material ¾ in. or less in thickness, around a pin the diameter of which is equal to the thickness of the specimen.

(*b*).—For material more than ¾ in. to and including 1¼ in. in thickness, around a pin the diameter of which is equal to twice the thickness of the specimen.

(*c*).—For material more than 1¼ in. in thickness, around a pin the diameter of which is equal to three times the thickness of the specimen.

911.—The test specimens for pins, rollers, and other bars, when prepared as specified in Article 916, shall bend cold through 180° around a 1-in. pin without cracking.

912.—The test specimens for rivet steel shall bend cold 180° flat on themselves without cracking.

913.—*Test Specimens.*—Test specimens shall be prepared for testing from the material in its rolled or forged condition, except when it is specified to be annealed, in which case the specimens shall be prepared from the material as annealed for use, or from a short length of a full section similarly treated. Test specimens shall be taken longitudinally and, except as specified in Articles 914, 915, and 916, shall be of the full thickness or diameter of material as rolled.

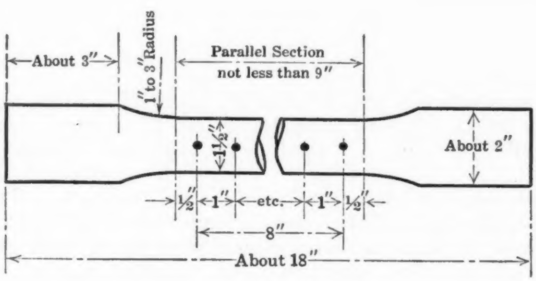


FIG. 4.

914.—Test specimens for plates, shapes, and flats may be machined to the form and dimensions shown in Fig. 4, or with both edges parallel; except that bend-test specimens for eye-bar flats may have three rolled sides. Tension-test specimens for plates and eye-bar flats over 1½ in. in thickness, and bend-test specimens for plates over 1½ in. in thickness, may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.

915.—Test specimens for bars over 1½ in. in thickness or diameter may be machined to a thickness or diameter of at least ¾ in. for a length of at least 9 in.; or tension-test specimens may conform to the dimensions shown in Fig. 5, in which case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens may be 1 in. by ½ in. in section.

916.—Tension-test specimens for pins and rollers shall conform to the dimensions shown in Fig. 5. In this case, the ends shall be of a form to fit the holders of the testing machine in such a way that the load shall be axial. Bend-test specimens shall be 1 in. by ½ in. in section. The tension-test specimen

shown in Fig. 5 and the 1 in. by ½-in. bend-test specimen for pins and rollers shall be taken so that the axis is 1 in. from the surface; and for other bars over 1½ in. in thickness or diameter, midway between the center and surface.*

917.—Tension and bend-test specimens for rivet steel shall be of the full-size sections of the bars as rolled. If the bars have been cold-drawn, they shall be normalized before testing.

918.—Number of Tests.—One tension and one bend test shall be made from each melt, except that if material from one melt differs ⅜ in. or more in thickness, one tension and one bend test shall be made from both the thickest and the thinnest material rolled.

919.—If any test specimen shows defective machining or develops flaws, it may be discarded and another specimen substituted.

920.—If the percentage of elongation of any tension-test specimen is less than that specified in Article 902, and any part of the fracture is more than ¾ in. from the center of the gauge length of a 2-in. specimen, or is outside the middle-third of the gauge length of an 8-in. specimen, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed.

921.—Character of Fracture.—Test specimens of structural or rivet steel shall show a fracture of uniform, silky appearance, of bluish gray or dove color, and entirely free from granular, black and brilliant specks.

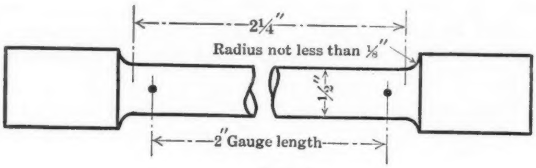


FIG. 5.

922.—Surface Defects.—Finished rolled material shall be free from cracks, flaws, injurious seams, blisters, ragged and imperfect edges, and other surface defects. It shall have a smooth finish, and shall be straightened in the mill before shipment.

923.—Permissible Variations in Weight and Thickness.—The cross-section or weight of each piece of steel shall not vary more than 2.5% from that specified, except in the case of sheared plates which shall be covered by the following permissible variations: One cubic inch of rolled steel is assumed to weigh 0.2833 lb.:

(*a*).—When ordered to weight per square foot, the weight of each lot in each shipment shall not vary from the weight ordered more than the amount given in Table 1. The term "lot", as applied to Table 1, means all the plates in each group width and group weight.

(*b*).—When ordered to thickness, the thickness of each plate shall not vary more than 0.01 in. under that ordered. The overweight of each lot in each shipment shall not exceed the amount given in Table 2. The term, "lot", as applied to Table 2, means all the plates of each group width and group thickness.

924.—Marking.—The name or brand of the manufacturer and the melt number shall be legibly stamped or rolled on all finished material, except that rivet and lacing-bars and other small sections, when loaded for shipment, shall be separated properly and marked for identification. The identification marks shall be stamped legibly on the end of each pin and roller. The melt number shall be marked legibly by stamping, if practicable, on each test specimen.

* The gauge length, parallel portions, and fillets shall be as shown in Fig. 5, but the ends may be of any form which will fit the holders of the testing machine.

TABLE 1.—PERMISSIBLE VARIATIONS OF PLATES ORDERED TO WEIGHT.*

Ordered weight, in pounds per square foot.	Permissible Variations in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Ordered Weights:																	
	Under 48 in.		48 to 60 in., excl.		60 to 72 in., excl.		72 to 84 in., excl.		84 to 96 in., excl.		96 to 108 in., excl.		108 to 120 in., excl.		120 to 132 in., excl.		132 in., or over.	
	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under	Over	Under
Under 5	5	3	5.5	3	6	3	7	3	....	....	....	....	....	....	....	....	....	....
5 to 7.5, excl	4.5	3	5	3	5.5	3	6	3	....	....	....	....	....	....	....	....	....	....
7.5 " 10, "	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	....	....	....	....
10 " 12.5, "	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3	9	3
12.5 " 15, "	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3	8	3
15 " 17.5, "	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3	7	3
17.5 " 20, "	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3	6	3
20 " 25, "	2	2	2.5	2	2.5	2.5	3	2.5	3.5	2.5	4	3	4.5	3	5	3	5.5	3
25 " 30, "	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3	5	3
30 " 40, "	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3	4.5	3
40, or over	2	2	2	2	2	2	2	2	2.5	2	2.5	2.5	3	2.5	3.5	3	4	3

* The weight per square foot of individual plates shall not vary from the ordered weight by more than one and one-third times the amount given in Table 1.

TABLE 2.—PERMISSIBLE OVERWEIGHTS OF PLATES ORDERED TO THICKNESS.*

Ordered thickness, in inches.	Permissible Excess in Average Weights per Square Foot of Plates for Widths Given, Expressed in Percentages of Nominal Weights:								
	Under 48 in.	48 to 60 in., excl.	60 to 72 in., excl.	72 to 84 in., excl.	84 to 96 in., excl.	96 to 108 in., excl.	108 to 120 in., excl.	120 to 132 in., excl.	132 in. or over
Under 1/8	9	10	12	14	....	....	....	....	....
1/8 to 3/16 excl	8	9	10	12	....	....	....	....	....
3/16 " 1/4 "	7	8	9	10	12	....	....	....	....
1/4 " 5/16 "	6	7	8	9	10	12	14	16	19
5/16 " 3/8 "	5	6	7	8	9	10	12	14	17
3/8 " 7/16 "	4.5	5	6	7	8	9	10	12	15
7/16 " 1/2 "	4	4.5	5	6	7	8	9	10	13
1/2 " 5/8 "	3.5	4	4.5	5	6	7	8	9	11
5/8 " 3/4 "	3	3.5	4	4.5	5	6	7	8	9
3/4 " 1 "	2.5	3	3.5	4	4.5	5	6	7	8
1, or over	2.5	2.5	3	3.5	4	4.5	5	6	7

* The weight of individual plates ordered to thickness shall not exceed the nominal weight by more than one and one-third times the amount given in Table 2.

B.—Cast Steel.

The specifications of the American Railway Engineering Association for cast steel as recommended in its *Proceedings*, Vol. 21 (1920), p. 518, agree fairly well with the specifications of the American Society for Testing Materials, A27-16, p. 220, of the Standards of 1918, so far as requirements for cast steel suitable for bridgework is concerned. The latter specifications cover several grades of castings, some of which are not suitable for bridges, while the specifications of the American Railway Engineering Association cover only bridge-work castings. The following specifications conform to the requirements of those of the American Railway Engineering Association.

927.—Process.—Cast steel shall be made by the open-hearth or the crucible process.

928.—Heat Treatment.—Castings shall be annealed.

929.—Properties.—Test specimens of cast steel shall conform to the following requirements as to chemical composition and tensile properties:

Elements considered.	Minimum tensile strength, in pounds per square inch.	Minimum yield point, in pounds per square inch.	Minimum elongation in 2 in.	Minimum reduction of area.
Phosphorus not more than 0.05%... Sulphur not more than 0.05%.......	60 000	30 000	22%	30%

930.—Ladle Analyses.—An analysis of each melt of steel shall be made by the manufacturer to determine the percentage of carbon manganese, phosphorus, and sulphur. This analysis shall be made from drillings taken at least ¼ in. beneath the surface of a test ingot obtained during the pouring of the melt. The chemical composition thus derived shall be reported to the Engineer.

931.—Check Analyses.—Check analyses may be made by the Engineer from a broken tension or bend-test specimen. The phosphorus and sulphur content thus derived shall not exceed that specified in Article 929 by more than 20 per cent. Drillings for analysis shall be taken not less than ¼ in. beneath the surface.

932.—Yield Point.—The yield point shall be determined by the drop of the beam of the testing machine.

933.—Speed of Testing Machine.—The cross-head speed of the testing machine shall be such that the beam of the machine can be kept in balance, but in no case shall the values given in the following table be exceeded.

Gauge length of specimen, in inches.	Maximum Cross-head Speed, in Inches per Minute, in Determining:	
	Yield point.	Tensile strength.
2	0.5	2.0
8	2.0	6.0

934.—Bend Test.—The test specimen shall bend cold through 120° around a 1-in. pin, without cracking.

935.—Test Specimens.—Sufficient test bars from which the test specimens required by Article 936 may be selected, shall be attached to castings weighing 500 lb., or more, when the design of the castings will permit. If the castings weigh less than 500 lb., or are of such design that test bars cannot be attached, two test bars shall be cast to represent each melt. Test bars shall be annealed with the castings they represent. Tension-test specimens shall conform to the dimensions shown in Fig. 5 of Structural Steel Specifications. Bend-test specimens shall be machined to 1 in. by ½ in. in section, with corners rounded to a radius of not more than $\frac{1}{16}$ in.

936.—Number of Tests.—One tension and one bend test shall be made from each annealing charge. If more than one melt is represented in the annealing charge, one tension and one bend test shall be made from each melt. If the percentage of elongation of any tension-test specimen is less than that specified in Article 929, and any part of the fracture is more than ¾ in. from the center of the gauge length, as indicated by scribe scratches marked on the specimen before testing, a re-test shall be allowed. If the results of the physical tests of any test lot do not conform to the requirements specified, the manufacturer may re-anneal such lot not more than twice, and re-tests shall be made as specified in Article 929.

937.—Workmanship and Finish.—The castings shall conform substantially to the drawings and shall be made in a workmanlike manner. The castings shall be free from injurious defects.

938.—Inspection at Foundry.—Tests and inspection shall be made at the place of manufacture before shipment, and shall be conducted so as not to interfere unnecessarily with the operation of the works.

939.—Rejection.—Castings which show injurious defects subsequent to their acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

C.—Cast Iron.

The specifications of the American Railway Engineering Association for cast iron, as recommended by its *Proceedings,* Vol. 21 (1920), page 519, are in accord with the specifications of the American Society for Testing Materials, Serial Number A48-18, page 406 of the Standards of 1918. The following specifications are in accord therewith.

940.—Excess.—Cast iron shall be of tough, gray iron, and shall be made by the cupola process.

941.—Workmanship and Finish.—Castings shall be true to pattern and free from excessive shrinkage. They shall be free from cracks, cold shuts, blowholes, and other flaws.

942.—Chemical Composition.—The sulphur content of cast iron shall not exceed the following:

Light castings....................	0.10 per cent.
Medium castings..................	0.10 per cent.
Heavy castings..................	0.12 per cent.

Drillings taken from the fractured ends of the transverse test bars shall be used for the sulphur determinations. One determination shall be made from each set of bars.

943.—Classification.—Castings shall be classified as light, medium and heavy:

(*a*).—Light castings are those having any section less than ½ in. thick.

(*b*).—Heavy castings are those having no section less than 2 in. thick.

(*c*).—Medium castings are those not included in either of the two classes above.

944.—Test Bar.—Tests shall be made on the "arbitration test bar" of the American Society for Testing Materials, as shown by Fig. 1, Serial A48-18.

945.—Tension Tests.—Tension tests shall be made only when specified by the Engineer and at the expense of the Purchaser.

946.—Number of Tests.—Two sets of two test bars each shall be cast from each melt in thoroughly dried, green sand moulds, one set from the first iron poured and the other set from the last iron poured. Where the melt exceeds 20 tons, an additional set of two bars shall be cast from each additional 20 tons, or fraction thereof.

947.—Transverse Tests.—A transverse test of each bar shall be made. The load shall be applied at the middle, and the supports shall be spaced 12 in. apart. The load on the test bar at rupture shall be not less than the following:

Light castings........................	2 500 lb.
Medium "	2 900 "
Heavy "	3 300 "

The deflection at rupture shall in no case be less than 0.10 in. The rate of application of the load shall be such that a central deflection of 0.10 is produced in from 20 to 40 sec.

948.—Inspection at Foundry.—Tests and inspection shall be made at the place of manufacture before shipment, and shall be conducted so as not to interfere unnecessarily with the operation of the works.

949.—Rejection.—Castings which show injurious defects subsequent to their acceptance at the manufacturer's works will be rejected, and the manufacturer shall be notified.

Section 10.—Workmanship.

1001.—General.—The workmanship and finish shall be equal to the best general practice in modern bridge shops.

1002.—Rolled material before being laid off or worked must be straight. If straightening or flattening is necessary, it shall be done by methods that will not injure the material. Sharp kinks or bends may be cause for rejection.

1003.—Class of Workmanship.—Two kinds of workmanship are covered by these specifications and the Engineer will determine which kind is required:

(*a*).—Full punched and riveted work: In this work the only reaming required is to make fair holes and to meet the requirements of Article 1004.

(*b*).—Sub-punched and reamed work or drilled work: In this work full punching is allowed only for minor parts.

1004.—Rivet Holes in Full Punched Work.—

(*a*).—Material not exceeding in thickness the nominal diameter of the rivet may be punched full size (except as modified in (*c*), provided it is part of a member composed of not more than five thicknesses and not exceeding $4\frac{1}{2}$ in. in total thickness.

(*b*).—If the limitations specified in (*a*) are exceeded, the material shall be drilled.

(*c*).—Holes for field connections, except those in lateral, longitudinal, and sway-bracing, shall be drilled or reamed in the shop with connecting parts assembled or, if permitted by the Engineer, drilled or reamed to metal templet. Holes in field connections of special bracing, such as skew portals, shall be drilled or reamed in the shop as indicated above.

1005.—Full Punching.—Holes punched and not reamed shall be $\frac{1}{16}$ in. larger than the nominal diameter of the rivets. The diameter of the die shall not exceed the diameter of the punch by more than $\frac{1}{16}$ in. The punching shall be done so accurately that, after assembling, the cold rivet may be entered in 75% of the holes without drifting. If any holes must be enlarged to admit the rivets, they shall be reamed. Holes must be clean cut, without torn or ragged edges.

1006.—Shearing in Full-Punched Work.—Material may be sheared to dimensions if the distance from the center of the rivet hole nearest the sheared edge is equal to or greater than one and three-quarter times the diameter of the rivet. If this distance is less than one and three-quarter times the diameter of the rivet, or if the material is more than $\frac{3}{4}$ in. thick and the sheared edges are in tension, the edges must be planed off $\frac{1}{4}$ in. Re-entrant cuts shall be filleted before cutting.

1007.—Sub-Punched and Reamed or Drilled Work.—In sub-punched work the diameter of the punch shall be $\frac{3}{16}$ in. less than the nominal diameter of the rivet, and the die must not be more than $\frac{3}{32}$ in. larger than the punch. Holes shall be reamed $\frac{1}{16}$ in. larger than the nominal diameter of the rivets. Where drilling is done after the pieces are assembled, the holes may be drilled full size. Holes in material $\frac{7}{8}$ in. thick and less used for lateral, longitudinal, and sway-bracing, lacing, stay-plates and diaphragms, may be punched full size. Holes in other material, $\frac{3}{4}$ in. thick and less, shall be sub-punched and reamed, and in material more than $\frac{3}{4}$ in. in thickness, shall be drilled.

1008.—Accuracy of Sub-Punching.—In sub-punched and reamed work, the punching shall be done so accurately that, after assembling and before reaming, a cylindrical pin, $\frac{1}{16}$ in. smaller in diameter than the nominal size of the punched hole, may be entered, perpendicular to the face of the member, without

drifting, in at least 65 of any group of 100 contiguous holes in the same plane, and so that a pin $\frac{1}{8}$ in. smaller in diameter than the nominal size of the punched hole may be entered similarly in at least 85 of such holes. If these requirements are not fulfilled, the badly punched pieces shall be rejected. If any hole will not pass a pin $\frac{3}{16}$ in. smaller in diameter than the nominal size of the punched hole, this may be cause for rejection.

1009.—Planing Edges in Reamed Work.—Sheared edges of material more than $\frac{5}{8}$ in. in thickness and carrying a calculated stress, shall be planed to a depth of $\frac{1}{4}$ in. Re-entrant cuts shall be filleted before cutting.

1010.—Match-Marking.—Connecting parts assembled in the shop for reaming or drilling field connection holes, shall be match-marked and a marking diagram shall be furnished the Engineer.

1011.—Rivets.—The size of rivets called for on the plans shall be the size before heating.

1012.—Rivet heads when not countersunk or flattened shall be of approved shape and uniform size for each diameter. The heads shall be full, neatly made, concentric with the rivet, and in full contact with the surface of the member.

1013.—Riveting.—Rivets shall be heated uniformly to a light cherry red and driven while hot in a manner to insure tightness. When ready for driving, they shall be free from slag or carbon deposits and, when driven, shall completely fill the holes. All loose, burned, or otherwise defective rivets shall be carefully removed so as not to injure the adjacent metal, and replaced with perfect rivets. If necessary, they shall be drilled out. Caulking and re-cupping will not be permitted. Rivets shall be driven, where practicable, by direct acting riveters which retain the pressure after upsetting is completed. When necessary to drive rivets with a pneumatic riveting hammer, a pneumatic bucker shall be used for holding up when practicable.

1014.—Field rivets shall be furnished in excess of the nominal number required to the amount of 15% plus 10 rivets for each size and length.

1015.—Turned Bolts.—Where turned bolts are used to transmit shear the holes shall be reamed parallel and the bolts shall make a tight fit with the threads entirely outside the holes. A washer not less than $\frac{1}{4}$ in. thick shall be used under each nut.

1016.—Web-Plates.—Web-plates of girders which have no cover-plates may be $\frac{1}{2}$ in. less in width than the distance back to back of flange angles. When web-plates are spliced, a clearance of not more than $\frac{3}{8}$ in. between ends of plates will be allowed.

1017.—Finishing Members.—All members shall be true to line, free from twists and bends, and of correct lengths. The field rivet holes shall match.

1018.—Screw Ends.—Screw threads shall make close fits in the ends and shall be U. S. Standard, except that for pin-ends of diameters greater than $1\frac{3}{8}$ in., they shall be made with 6 threads per inch.

1019.—Welds.—Welds will not be allowed, except for the purpose of remedying minor defects in steel castings.

1020.—Bearing Surfaces.—The top and the bottom surfaces of base and cap plates of columns and pedestals, except where they are in contact with masonry, shall be planed or hot-straightened, and the parts of members in contact with them shall be faced to fit. Sole plates and masonry plates for plate girders shall be planed or hot-straightened and the sole plates shall have full contact with the girder flanges. Cast pedestals shall be planed on the surfaces in contact with the steel and shall have surfaces resting on masonry rough finished.

1021.—Fit of Stiffeners.—Stiffeners under the top flanges of deck girders and at all bearing points shall be milled or ground to bear against flange

angles. Other stiffeners must fit sufficiently tight against flange angles to exclude water after painting. Fillers and splice-plates shall fit within $\frac{1}{4}$ in. at each end.

1022.—Facing Ends.—Floor-beams, stringers, and girders having end-connection angles shall be faced to exact length after the connection angles are riveted. The thickness of the angles shall not be reduced more than $\frac{1}{8}$ in. at any point.

1023.—Abutting Joints.—Abutting joints in compression members and girder flanges and where specified in tension members of trusses shall be faced and brought to an even bearing. Where joints are not faced, the openings shall not exceed $\frac{1}{4}$ in.

1024.—Eye-Bars.—Eye-bars shall be straight, true to size, and free from twists, folds in the neck or head, and other defects. The heads shall be made by upsetting, rolling, or forging. Welding will not be allowed. The form of the heads will be determined by the dies in use at the works where the eye-bars are made, if satisfactory to the Engineer, but the manufacturer shall guarantee the bars to break in the body under the requirements of the full-size tests. The thickness of the head and neck shall not over-run more than $\frac{1}{16}$ in. for bars 8 in. or less in width, $\frac{1}{8}$ in. for bars more than 8 in. and not more than 12 in. in width, and $\frac{3}{16}$ in. for bars more than 12 in. in width.

1025.—Eye-bars of the same length and size of pin-hole shall be bored so accurately that on being placed together the pins shall pass through the holes at both ends at the same time without driving. Pin-holes at both ends shall be bored at the same time.

1026.—Annealing.—

(*a*).—Eye-bars shall be annealed by heating uniformly to the proper temperature followed by slow and uniform cooling. Proper instruments shall be provided for determining at all times the temperature of the bars.

(*b*).—Steel castings and all steel members which have been partly heated, except crimped stiffeners and minor parts, shall be properly annealed.

1027.—Pin Clearances.—For pins 5 in. or less in diameter, pin-holes shall be the diameter of the pin plus $\frac{1}{50}$ in. For pins more than 5 in. in diameter, the pin-holes shall be the diameter of the pin plus $\frac{1}{32}$ in.

1028.—Boring Pin-Holes.—Pin-holes shall be bored true to gauge, smooth, straight, parallel, and at right angles with the axis of the member, unless otherwise required. The variation from the specified distance from outside to outside of pin-holes in tension members, and from inside to inside of pin-holes in compression members, shall not exceed $\frac{1}{32}$ in. In built-up members, the boring shall be done after the member is riveted.

1029.—Boring Pins.—Pins 9 in. or more in diameter shall have a hole not less than 2 in. in diameter, bored longitudinally through the center.

1030.—Pins and Rollers.—Pins and rollers shall be accurately turned to gauge and shall be straight, smooth, and free from flaws.

1031.—Forging Pins.—Pins more than 7 in. in diameter shall be forged and annealed.

1032.—Pilot Nuts.—Two pilot nuts and two driving nuts shall be furnished for each size of pin, unless otherwise specified.

Section 11.—Weighing and Shipping.

1101.—Weight Paid For.—Payment on pound-price contract shall be based on scale weight of material in the fabricated structure, including field rivets.

1102.—Variation of Weight.—

(*a*).—If the weight of any member is more than $2\frac{1}{2}\%$ less than the computed weight, it may be cause for rejection.

(*b*).—The greatest allowable variation of the total scale weight of any structure from the weights computed from the approved shop drawings shall be $1\frac{1}{2}$ per cent. Any weight in excess of $1\frac{1}{2}\%$ above the computed weight shall not be paid for.

1103.—Computed Weight.—

(*a*).—The weight of steel shall be assumed at 0.2833 lb. per cu. in.; that of cast iron at 0.26 lb. per cu. in.

(*b*).—The weight of rolled shapes and of plates, up to and including 36 in. in width, shall be computed on the basis of their nominal weights and dimensions, as shown on the approved shop drawings, deducting for copes, cuts, and open holes.

(*c*).—The weights of plates more than 36 in. in width shall be computed on the basis of their dimensions as shown on the approved shop drawings. To this weight shall be added one-half the percentage for variation given in Article 923, Section 9, Material Specifications.

(*d*).—The weights of castings shall be computed from the dimensions shown on approved shop drawings, with an addition of 10% for fillets and over-run.

1104.—Weighing of Members.—Finished work shall be weighed in the presence of the Inspector, if practicable. The Contractor shall furnish satisfactory scales and do the handling of the material for weighing.

1105.—Marking and Shipping.—

(*a*).—Members weighing more than 5 tons shall have the weight marked thereon. Bolts and rivets of one length and diameter, also loose nuts or washers of each size, shall be packed separately. Pins, small parts, small packages of bolts, rivets, washers, and nuts, shall be shipped in boxes, crates, kegs, or barrels, the gross weight of any one of which shall not exceed 300 lb. A list and description of the contents shall be plainly marked on each package, box, or crate.

(*b*).—Long girders shall be loaded and marked so that they may arrive at the bridge site in position for erection without turning.

(*c*).—Anchor-bolts, washers, and other anchorage or grillage materials, shall be shipped to suit the requirements of the masonry construction.

Section 12.—Shop Painting.

1201.—Shop Cleaning and Painting.—Unless otherwise specified, steelwork, after it has been accepted by the Inspector and before it has left the shop, shall be thoroughly cleaned and given one coat of approved paint, applied in a workmanlike manner and well worked into the joints and open spaces. Cleaning shall be done with steel brushes, hammers, scrapers, and chisels, or by other equally effective means. Oil, paraffin, and grease shall be removed by wiping with benzine or gasoline. Loose dirt shall be brushed off with a dry bristle brush before the paint is applied.

1202.—Surfaces in Contact.—Surfaces coming in contact shall be cleaned and given one coat of linseed oil on each surface before assembling.

1203.—Erection Marks.—Erection marks shall be painted on painted surfaces.

1204.—Painting in Damp or Freezing Weather.—Painting shall not be done in damp or freezing weather, except under cover, and the steel must be free from moisture or frost when the paint is applied. Material painted under cover in damp or freezing weather shall be kept under cover until the paint is dry.

1205.—Mixing of Paint.—Paint shall be thoroughly mixed before applying, and the pigments shall be kept in suspension.

1206.—Machine-Finished Surfaces.—Machine-finished surfaces of steel (except abutting joints and base-plates), shall be coated with white lead and tallow, applied hot, as soon as the surfaces are finished and accepted by the Inspector.

SECTION 13.—MILL AND SHOP INSPECTION.

1301.—Facilities for Inspection.—The Contractor shall allow the Inspector full access to necessary parts of the premises and shall furnish all facilities for inspection of material and workmanship in the mill and shop.

1302.—Mill Orders and Shipping Statements.—The Contractor shall furnish the Engineer with as many copies of mill orders and shipping statements as the Engineer may direct. The shipping statements shall show weights of individual members.

1303.—Notice to Engineer.—The Contractor shall give ample notice to the Engineer of the beginning of rolling and of the shop work, so that inspection may be provided. No material shall be rolled or work done before the Engineer has been notified where the orders have been placed.

1304.—Cost of Testing.—The Contractor shall furnish, without charge, test specimens, as specified herein, and all labor, testing machines, and tools necessary to make the specimen and full-size tests.

1305.—Inspector's Authority.—The Inspector shall have the power to reject materials and workmanship which do not come up to the requirements of these specifications; but, in case of dispute, the Contractor may appeal to the Engineer, whose decision shall be final.

1306.—Rejections.—

(*a*).—The acceptance of any material or finished members by the Inspector shall not be a bar to their subsequent rejection, if found defective.

(*b*).—Rejected material and workmanship shall be replaced promptly or made good by the Contractor.

SECTION 14.—FULL-SIZE TESTS.

1401.—Eye-Bar Tests.—

(*a*).—The number and size of the bars to be tested shall be stipulated by the Engineer before the mill order is placed. The number shall not exceed 5% of the whole number of bars ordered, with a minimum of two bars on small orders.

(*b*).—The test bars shall be of the same section as the bars to be used in the structure and of the same length, if within the capacity of the testing machine. They shall be selected by the Inspector from the finished bars preferably after annealing. Test bars representing bars too long for the testing machine shall be selected from the full-length bar material after the heads on one end have been formed and shall have the second head formed on them after being cut to the greatest length which can be tested.

(*c*).—Full-size tests of eye-bars shall show a yield point of not less than 30 000 lb. per sq. in., an ultimate strength of not less than 55 000 lb. per sq. in., and an elongation of not less than 10% in a length of 20 ft. measured in the body of the bar. The fracture shall show a silky or fine granular structure throughout.

(*d*).—If a bar fails to meet the requirements of Article 1401 (*c*), two additional bars of the same size and from the same mill heat shall be tested. If the failure of the first test bar is on account of the character of the fracture only, the bars represented by the test may be re-annealed before the additional bars are tested.

(*e*).—If two of the three bars tested fail, the bars of that size and mill heat shall be rejected.

(*f*).—A record of the annealing charges shall be furnished the Engineer showing the bars included in each charge and the treatment they receive.

(*g*).—Bars thus tested which meet the requirements of the specifications shall be paid for by the Purchaser at the same unit price as for the structure. Bars which fail to meet the requirements of the specifications, and all bars rejected as a result of tests, shall be at the Contractor's expense.

The Special Committee on Specifications for Bridge Design and Construction,

H. B. SEAMAN, *Chairman,*
H. C. BAIRD, *Secretary,*
C. W. HUDSON,
M. S. KETCHUM,
B. R. LEFFLER,
A. F. ROBINSON,
F. E. TURNEAURE,
J. R. WORCESTER.

NOVEMBER 16TH, 1921.

DISCUSSION

HENRY B. SEAMAN,* M. AM. SOC. C. E. (by letter).†—The Tentative Specifications for the Design and Construction of Steel Railroad Bridges which are offered for "full, exhaustive, and constructive discussion", "from all sources having experience and judgment on the subject", are the result of careful study by the Committee. They are presented in concise form in order that the discussion may be systematic and constructive; and in refraining, in the report, from expressing the many divergent views held by the members of the Committee, it was believed that the results thus far attained would be free from uncertainty and harmful ambiguity. It was deemed preferable that each member of the Committee should offer individual discussion, rather than present a minority report on a specification which is purely tentative. Yet, it is thought that the membership of the Society, and others, should be informed of the different aspects of the subject, which were presented before the Committee, in order that those who desire to contribute to the specification may join fully in its progress.

Suggestions on the general form of the specification undoubtedly will be received, either as regards the arrangement of clauses or the subject-matter; and also whether such a specification from the Society should enter into the same exhaustive detail followed by other organizations, or should confine itself to general principles, and thus permit preference to the individual engineer in minor details.

Several of the larger matters of general importance, having great influence in the design, such as engine loading, impact, and column formula, have received prolonged consideration by the Committee, and it seems proper that these subjects should be presented more fully than is shown in the mere outline of the Tentative Specification.

Engine Loading.—The subject of engine loading was considered from various standpoints. It was recognized that engine concentrations had increased far beyond the loads for which this distribution was intended originally, and that although this typical engine had simplified the calculations for design, a still further simplification was desirable, if practicable. To this end was suggested a uniform load with single concentration, and other methods, but when all things were considered, particularly the established tables and diagrams now in use, it was evident that there were no advantages over the present general practice. Furthermore, a study of the chart of engine moments and shears, as already considered by the American Railway Engineering Association,‡ showed that the typical engine distribution, even with the increased loading, gave satisfactory results as compared with the group of engines in actual service on several railroads. Whether it may yet be possible to devise a new system of loading which will conform even more closely to the engines in actual use, may be a subject for broad discussion. However, with this heavy loading, and the further provision in the specifications for overload,

* New York City.

† Received by the Secretary, November 10th, 1921.

‡ *Proceedings*, Am. Ry. Eng. Assoc., Vol. 21, p. 571.

it may be questioned whether it is still necessary to add the double concentration of passenger engines. This is particularly true if proper provision is made beyond 100%, for impact on spans of less than 30 ft.

Impact Formula.—The first impact formula which came to the writer's attention was presented by the late C. C. Schneider, Past-President, Am. Soc. C. E., in his specification of 1887, in which $I = 0.7 + \frac{5}{L}$. This formula was introduced to simplify the design of bridges, as compared with the application of the Launhardt formula which at that time was coming into general use. Eight years later (1895), the Schneider formula was replaced by that in which $I = \frac{300}{L + 300}$, sometimes known as the "Pencoyd formula," which was introduced into the bridge specifications of the American Railway Engineering and Maintenance of Way Association of 1905, 1906, and 1910.

The subject of impact has received more consideration and prolonged discussion by the Committee than any other item of the specification. The results of the tests* made by J. E. Greiner, M. Am. Soc. C. E., on the Baltimore and Ohio Railroad Bridges, of those† made by F. E. Turneaure, M. Am. Soc. C. E., and of those‡ made more recently in England by Maj. A. Mount, R. E., were before the Committee.

The tests of the American Railway Engineering and Maintenance of Way Association, unfortunately, did not include spans less than 35 ft. in length, but the results given by Maj. Mount, shown in Fig. 6, largely supply the deficiency, although the English engine, with its inside connections, may not be directly comparable with the American engine which has its connections outside the driving wheels and, possibly, gives greater impact from unbalanced forces. In spite of this favorable consideration of the English engine, it is noticed that in Maj. Mount's results a rail bearer, composed of two halves fastened together, of 7 ft. 6 in. loaded length, gave an impact of 159% on one-half and 90% on the other half, making an average of 125%; also a loaded length of 9 ft. 8 in. gave an extreme impact of 143%, with an average of 92 per cent. Such results are rather startling to those who have made no provision beyond 100% in very short spans, but they are what should have been expected from the hammer blow of unbalanced drivers on short, rigid beams. It approaches the hypothetical condition of the "irresistible force" meeting the "immovable body". Such short spans are rarely used and involve little extra expense, but proper provision should be made for the condition. The tendency toward a sharp increase of impact in very short spans is recognized to some extent by every impact formula that has been proposed, except the one recently adopted by the American Railway Engineering Association, in its 1920 specifications, namely, $I = \frac{300}{300 + \frac{L^2}{100}}$, which gives practically the same impact for all spans under 25 ft. The Committee recognized the advisa-

* *Proceedings*, Am. Ry. Eng. and M. of W. Assoc., Vol. 6 (1905).

† *Proceedings*, Am. Ry. Eng. and M. of W. Assoc., Vol. 12, Pt. 3 (1911).

‡ *Engineering News-Record*, October 20th, 1921.

bility of providing for greater increase of impact on short spans—125% was incidentally mentioned for 0 span, and 100% for spans of 30 ft.—but the new impact formula of the American Railway Engineering Association was introduced into the Tentative Specification which is proposed for discussion.

The tests show that impact for long spans—700 ft., or more—may be considered to be practically negligible. Furthermore, it is desirable to adopt a formula which will be applicable to spans of all lengths. There is no reason why the change from minimum impact to maximum impact should not be gradual and continuous, conditions do not reverse for short spans; but it must be remembered that the whole subject is empirical, and any formula adopted now may be modified by future experiments.

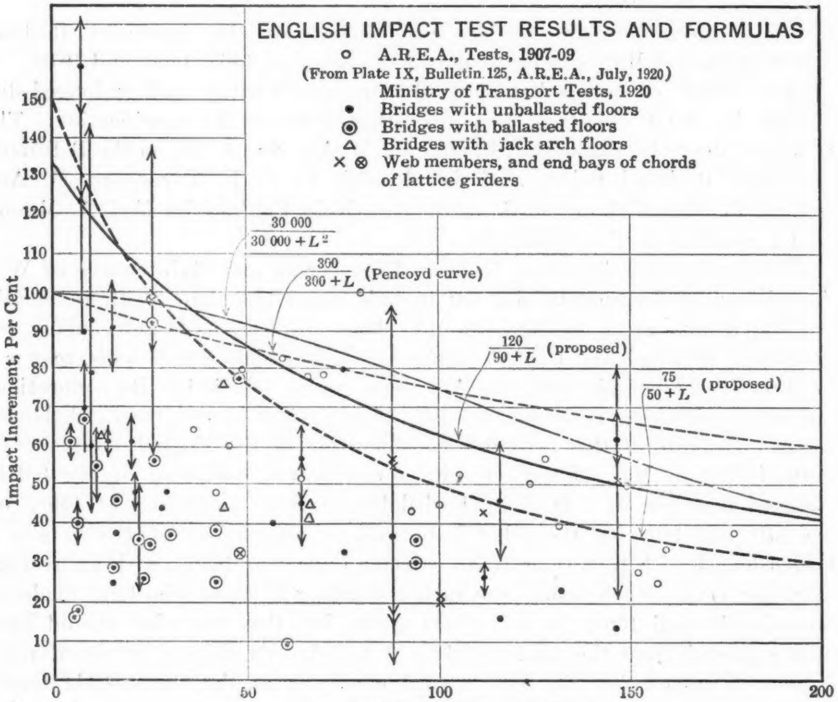


FIG. 6.

About 1905 it was the custom to use special specifications for each bridge designed, conforming to the particular conditions of span and loading. There were three or four different specifications for short spans, and another entirely different specification for long spans. There seemed to be no sharp line of demarcation between short and long spans, although it was generally assumed to be about 300 ft., at which the dead load approached or exceeded the live load. A single standard specification which would be applicable to spans of all lengths seemed to be eminently desirable. The fundamental difference in the design of short and long-span bridges was the relative action of the live

load, and this should be stated in a "formula for equivalent static stress", as it was then expressed. With the use of such a formula all live load stresses might be reduced to equivalent static stresses and the allowable unit stresses modified accordingly.

At that time the impact tests of the American Railway Engineering and Maintenance of Way Association had not been made, and the only guide for such a formula were the tests made by Mr. Greiner, previously mentioned, and the weights of bridges which had been built. The fact that impact on long spans would be negligible was already evident from the long spans erected in New York City, and has since been indicated by the tests; while for very short spans, impact of 100%, or more, seemed to be advisable. The formula, $I = \frac{300}{L + 300}$, already in use, was not applicable to very long spans, and the formula of the quarter ellipse, $I = 125 - \frac{1}{8} \sqrt{2\,000\,L - L^2}$, was finally adopted. This formula was presented before the Society in 1912.*

The tests of the American Railway Engineering and Maintenance of Way Association, as published in its *Proceedings* for 1911, constituted a striking confirmation of this formula, as shown by Plate X† of the discussion of the writer's paper by S. W. Bowen, Assoc. M. Am. Soc. C. E. A more detailed examination of several of the tests, however, indicates further confirmation than was noticed in the discussion. Referring to the 1911 Report it will be noticed (Plate III-*bs*) that the 37 ft. 0-in. span indicates a maximum impact of 125%, which was so abnormally high that the result was considered to be unreliable. The next result in this set of tests is 92%, which is close to the elliptical curve. Again (Plate III-*bk*), for the 59 ft. 2-in. span, the maximum impact is 133%, which is also abnormal, and the next test of that set gives 82%, which also falls near the ellipse.

In Table 6–o the north girder of the 80-ft. span shows an impact of 122%, and the south girder shows 74%; the former again seems to be abnormal, while the latter comes near the elliptical curve. These three tests which were not plotted in the original diagram, are shown by circles (o) (Figs. 7 and 8). This curve runs from 0% for 1 000-ft. spans to 125% for 0 span, and provides for impact somewhat greater than 100% for spans less than 20 ft. It is the oldest impact formula which is applicable to spans of all lengths and deserves consideration unless something better is offered.

The impact formula offered by Maj. Mount, namely, $I = \frac{120}{90 + L}$, gives very satisfactory results for short spans, and for spans less than 30 ft., makes proper provision of more than 100% impact; but it seems to be too high for very long spans. It is interesting to note on Fig. 8, how the modified elliptical curve, $I = 135 - \frac{1}{6} \sqrt{1\,620\,L - L^2}$, conforms with the impact tests.

In selecting an impact formula, it should be remembered that a stress of 24 000 lb. per sq. in. is provided in cases of overload, and any formula which

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 340.

† *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 355.

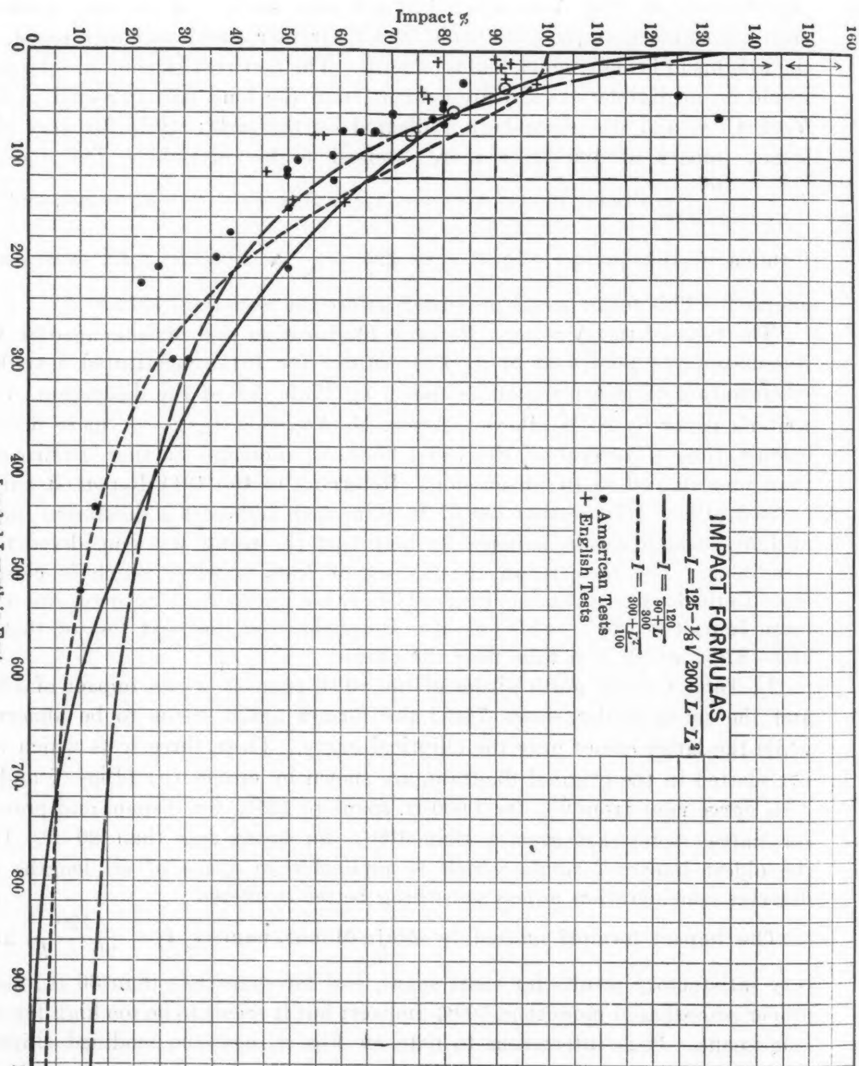
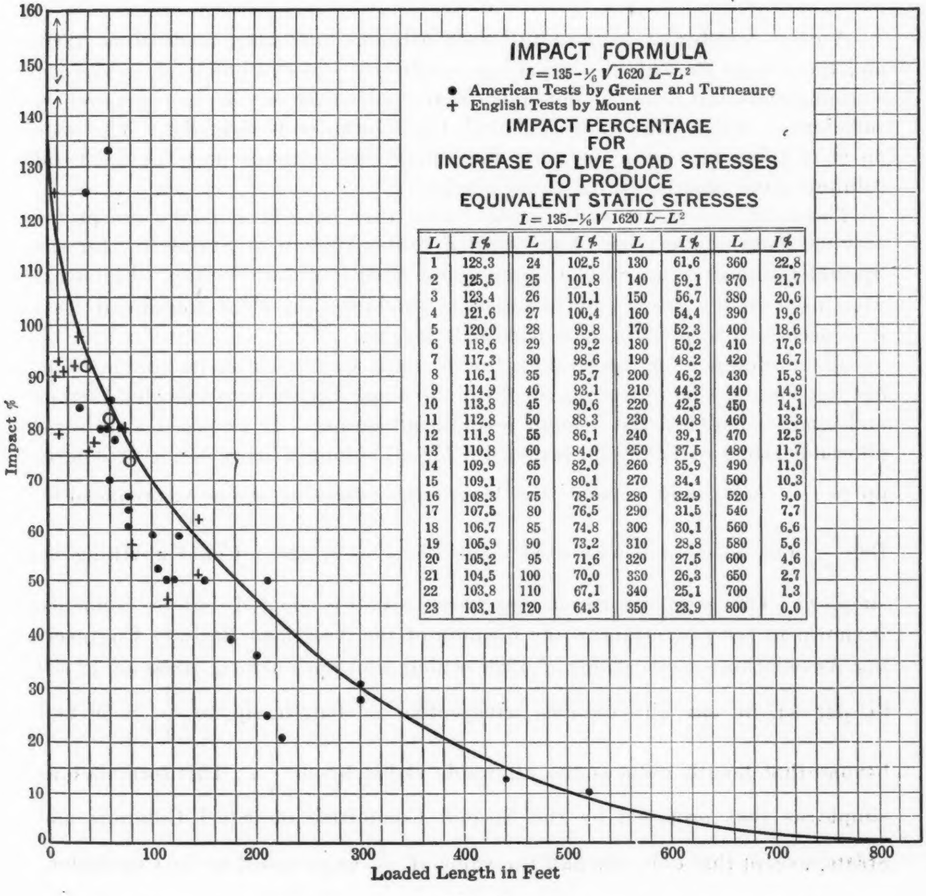

Impact %
IMPACT FORMULAS
$I = 125 - \frac{1}{8}\sqrt{2000\,L - L^2}$
$I = \frac{120}{90+L}$
$I = \frac{300}{300+\frac{L^2}{100}}$
• American Tests
+ English Tests
Loaded Length in Feet

Fig. 7.

may be adopted, therefore, should cover extreme conditions. To be able to maintain a bridge intelligently is quite as important as to construct one; in fact, engineers should always bear in mind the contingency of maintenance in bridge design.

Column Formula.—In the early days of iron bridges the practice was to proportion columns by one of three formulas, depending on the assumed condition of the column ends, whether fixed at both ends, fixed at one end



L	I%	L	I%	L	I%	L	I%
1	128.3	24	102.5	130	61.6	360	22.8
2	125.5	25	101.8	140	59.1	370	21.7
3	123.4	26	101.1	150	56.7	380	20.6
4	121.6	27	100.4	160	54.4	390	19.6
5	120.0	28	99.8	170	52.3	400	18.6
6	118.6	29	99.2	180	50.2	410	17.6
7	117.3	30	98.6	190	48.2	420	16.7
8	116.1	35	95.7	200	46.2	430	15.8
9	114.9	40	93.1	210	44.3	440	14.9
10	113.8	45	90.6	220	42.5	450	14.1
11	112.8	50	88.3	230	40.8	460	13.3
12	111.8	55	86.1	240	39.1	470	12.5
13	110.8	60	84.0	250	37.5	480	11.7
14	109.9	65	82.0	260	35.9	490	11.0
15	109.1	70	80.1	270	34.4	500	10.3
16	108.3	75	78.3	280	32.9	520	9.0
17	107.5	80	76.5	290	31.5	540	7.7
18	106.7	85	74.8	300	30.1	560	6.6
19	105.9	90	73.2	310	28.8	580	5.6
20	105.2	95	71.6	320	27.5	600	4.6
21	104.5	100	70.0	330	26.3	650	2.7
22	103.8	110	67.1	340	25.1	700	1.3
23	103.1	120	64.3	350	23.9	800	0.0

Fig. 8.

and round at the other, or round at both ends. Later, it was considered advisable to use only one formula, that for both ends round, since the end condition of columns in a structure are uncertain, and the column may be subjected to many unforeseen conditions. For columns with round ends the formula then used was of the Rankine-Gordon type, which was considered to be so approximate, and yet so safe, that a straight-line formula might give equally satisfactory results within the limits of column length usually found in

bridge design. It was a new thought, a change from the monotonous routine of years, and a recognition that engineers need not be confined to exact proportioning where unknown conditions were so prevalent. Tabulated values of r for various sections simplified its application, and the greater conveniences of the older formula—of the Rankine-Gordon type—were soon forgotten. The straight-line formula, introduced into the specifications of the American Railway Engineering and Maintenance of Way Association, of 1905, 1906, and 1910, was $p = 16\,000 \text{ lb.} - 70 \frac{l}{r}$.

A great number of tests of full-sized columns have been made since 1880, and these tests showed the surprising result that pin-end columns do not act as round ends, but rather as flat ends—that the bearing on the pin is apparently sufficient to fix the direction as though the column ends were flat. The tests on steel tubes also indicated that the square-end columns and the fixed-end columns gave practically the same results.

The most systematic and complete tests that have been made, except for extreme lengths, are those of the United States Bureau of Standards, for the Special Committee on Steel Columns and Struts of the Society. Flat ends were used in all cases, and in order to produce equivalent round-end tests it is necessary to plot the results to half length.

The preference for the straight-line formula was based on its simple form, but the Special Committee on Steel Columns and Struts went one step farther and almost abolished the column formula altogether. It proposed a uniform allowable stress of 12 000 lb. per sq. in. for all columns, irrespective of length, up to $\frac{l}{r} = 80$, and beyond that length the allowable stress was reduced by $100 \frac{l}{r}$, and no column was used longer than $\frac{l}{r} = 120$. Could anything be simpler? This simple formula was never actually adopted, but its influence is shown in the new straight-line formula of the American Railway Engineering Association specifications of 1920, which allows a uniform stress of 12 500 lb. per sq. in. on all columns, irrespective of length, up to $\frac{l}{r} = 50$, and beyond that length reduces the allowable stress by $50 \frac{l}{r}$. This formula is as simple as that proposed by the Special Committee on Steel Columns and Struts, except that only one-half the value of $\frac{l}{r}$ as proposed by the Committee, is deducted for long columns. Both these formulas neglect the well established rule that the maximum allowable column stress should be placed at $\frac{l}{r} = 40$, which corresponds to $\frac{l}{d} = 12$ for solid rectangular sections. The upper limit of 12 500 lb., as compared with the basis of 16 000 lb. for tension, seems, however, a useless waste of material, and its application may involve needless expense to railroads. It will condemn many existing bridges which have been well designed under the previously accepted column formulas.

It is unscientific, and particularly objectionable in long spans where all useless dead weight should be eliminated. The confidence in a straight-line formula appears to be wavering. The Canadian Engineering Standards Association has abandoned it altogether and adopted the "parabolic formula", $I = 12\,500 - \frac{1}{4}\,\frac{l^2}{r^2}$, which, although it is as purely empirical as the straight-line formula, may be used in maintenance for bridge rating. Is it not an opportune time for engineers to return to a reconsideration of the entire subject? Each individual may formulate his own practice, but a specification should be based on scientific principles.

The Euler formula for stress due to flexure in long columns has remained unquestioned for 160 years, the only criticism being that it does not provide for the direct stress to which all columns are subjected. We cannot do better than accept this as a basis for a formula for combined stresses.

The substance of the following clause is found in every bridge specification:

"Members subject to both direct and bending stresses shall be proportioned so that the combined fiber stress shall not exceed the allowable stress specified."

Long columns, at the instant of rupture, are subjected to both direct and bending stresses. The direct stress is proportional to the applied load, P. The bending stress, according to the Euler formula, is inversely proportional to $\frac{l^2}{r^2}$; thus according to Euler:

$$P = A\,\pi^2\,E\left(\frac{r}{l}\right)^2 = a\left(\frac{r}{l}\right)^2 = \frac{a}{\frac{l^2}{r^2}} = \frac{1}{\frac{l^2}{a\,r^2}}$$

The formula for the combined stresses may then be written:

$$Y = P + P\left(\frac{l^2}{a\,r^2}\right) = P\left(1 + \frac{l^2}{a\,r^2}\right) \quad\quad (1)$$

from which may be found the theoretical fiber stress in the column for any given load, P. This is of great importance in rating the strength of old bridges in service, and is invaluable to the maintaining engineer.

From Equation (1) the modified Euler column formula is derived, which includes provision for direct stress:

$$P = \frac{Y}{1 + \frac{l^2}{a\,r^2}} \quad\quad (2)$$

where P is the applied load, Y is the yield point of the material at the outer fiber, and a is a constant to be found by experiment.

In long columns the yield point of the material is the ultimate strength of the column since, at the instant of yielding, the flexure increases rapidly and the resulting increase of the bending moment in the column will cause complete failure if the testing machine will follow with full load as rapidly as the column collapses.

This formula has been in use since the early days of iron bridge design. It is applicable to columns of all lengths and may be used in maintenance as well as in construction.

For convenience in plotting the allowable stresses on columns by Equation (2), Table 3 may be used.

TABLE 3.—WORKING STRESSES FOR COLUMNS.

$$p = \frac{16.0\ k}{1 + \dfrac{l^2}{13\,500\ r^2}}$$

$\frac{l}{r}$	p	$\frac{l}{r}$	p	$\frac{l}{r}$	p	$\frac{l}{r}$	p	$\frac{l}{r}$	p
40	14.30 k	68	11.92 k	96	9.51 k	124	7.48 k	185	4.53 k
42	14.15 "	70	11.74 "	98	9.35 "	126	7.35 "	190	4.35 "
44	13.99 "	72	11.56 "	100	9.19 "	128	7.23 "	195	4.19 "
46	13.83 "	74	11.38 "	102	9.04 "	130	7.11 "	200	4.04 "
48	13.67 "	76	11.21 "	104	8.88 "	135	6.81 "	205	3.89 "
50	13.50 "	78	11.03 "	106	8.73 "	140	6.52 "	210	3.75 "
52	13.33 "	80	10.86 "	108	8.58 "	145	6.25 "	215	3.62 "
54	13.16 "	82	10.68 "	110	8.43 "	150	6.00 "	220	3.49 "
56	12.98 "	84	10.51 "	112	8.29 "	155	5.76 "	225	3.37 "
58	12.81 "	86	10.34 "	114	8.15 "	160	5.52 "	230	3.25 "
60	12.63 "	88	10.17 "	116	8.01 "	165	5.30 "	235	3.14 "
62	12.45 "	90	10.00 "	118	7.87 "	170	5.09 "	240	3.04 "
64	12.27 "	92	9.83 "	120	7.74 "	175	4.89 "	245	2.94 "
66	12.10 "	94	9.67 "	122	7.61 "	180	4.71 "	250	2.84 "

NOTE.—1 k. = 1 000 lb.

The design and construction of steel bridges is still in a transition stage. The loading has been increased until it would seem that the limit had been reached, but improvements in the material are still to be expected. Steel 50% stronger than the structural steel usually specified is already in use, and there is a possibility, even a probability, that a still stronger steel may soon be offered. When this comes the formula previously mentioned will remain unchanged, except that the numerator will be increased in proportion to the increased yield point of the new material.

The results of full-sized column tests made by the Bureau of Standards for the Special Committee on Steel Columns and Struts are plotted on Fig. 9, on which also is shown the comparative value of the old and the new straight-line formulas of the American Railway Engineering Association and of the modified Euler formula with relation to those tests. The coefficient, a, of the last named formula has been placed at 13 500.

The accuracy and value of the modified Euler formula need no explanation. It is the Rankine-Gordon formula of long service. Of the two American Railway Engineering Association formulas, the old one, $p = 16\ 000\ \text{lb.} - 70\ \frac{l}{r}$, is parallel in direction to the averages of the tests, and with a slight increase of the factor of safety would practically conform to those tests; while the new formula of 1920, $I = 12\ 500 - 50\ \frac{l}{r}$, allows greater stresses for the longer columns. Neither of these straight-line formulas can be used definitely to obtain the fiber stress to which a long column may be subjected in service. The only objection to the modified Euler formula seems to be that, for convenient use, it requires a table of values.

In Table 4 is given values of the coefficient $\left(1 + \frac{l^2}{13\,500\, r^2}\right)$, Equation (1) for values of $\frac{l}{r}$ from 40 to 250.

TABLE 4.

$\frac{l}{r}$	Coefficient.	$\frac{l}{r}$	Coefficient.	$\frac{l}{r}$	Coefficient.	$\frac{l}{r}$	Coefficient.	$\frac{l}{r}$	Coefficient.
40	1.118	68	1.343	96	1.683	124	2.139	185	3.535
42	1.131	70	1.363	98	1.711	126	2.176	190	3.674
44	1.143	72	1.384	100	1.741	128	2.214	195	3.817
46	1.157	74	1.406	102	1.771	130	2.252	200	3.963
48	1.171	76	1.428	104	1.801	135	2.350	205	4.113
50	1.185	78	1.451	106	1.832	140	2.452	210	4.267
52	1.200	80	1.474	108	1.864	145	2.558	215	4.424
54	1.216	82	1.498	110	1.896	150	2.667	220	4.585
56	1.232	84	1.523	112	1.929	155	2.780	225	4.750
58	1.249	86	1.548	114	1.963	160	2.896	230	4.919
60	1.267	88	1.574	116	1.997	165	3.017	235	5.091
62	1.285	90	1.600	118	2.032	170	3.141	240	5.267
64	1.303	92	1.627	120	2.067	175	3.269	245	5.446
66	1.323	94	1.655	122	2.103	180	3.400	250	5.630

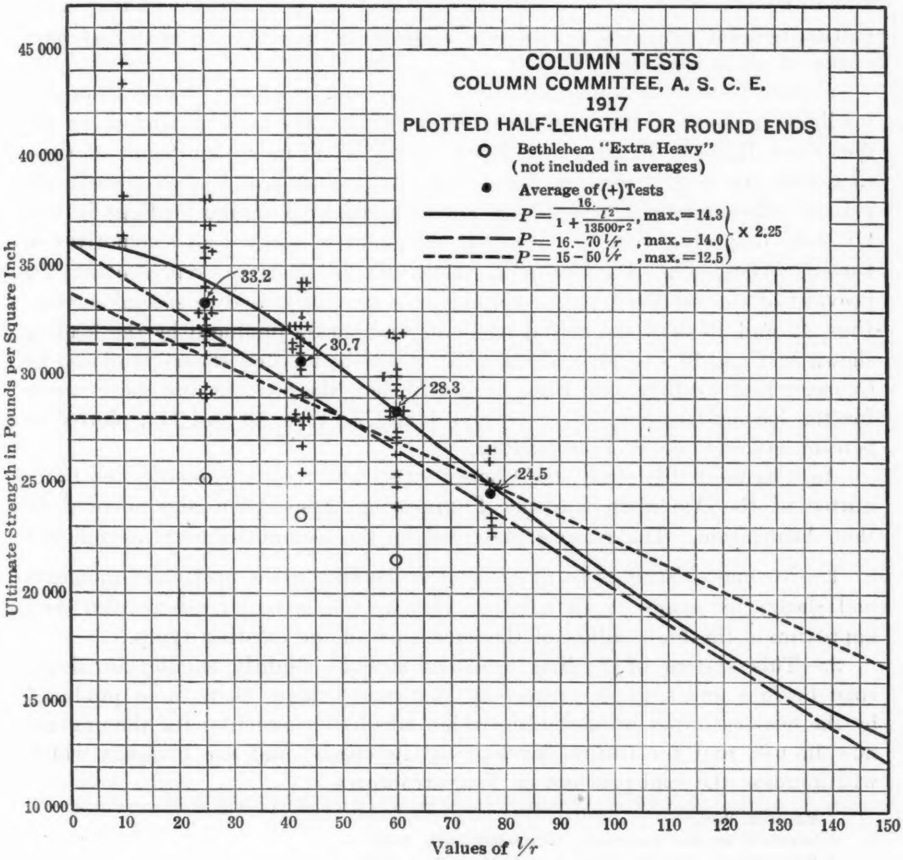


FIG. 9.

In presenting these specifications for discussion, the Special Committee on Specifications for the Design and Construction of Bridges has in mind the instruction of the Board of Direction to "confer and co-operate with similar committees of the American Railway Engineering Association and the Engineering Institute of Canada". As seven of the nine members of this Committee are members of the American Railway Engineering Association, and three of these are also members of the committee of that Association, there can be no question but that the work of the Association has received full consideration. The Canadian specifications were before the Committee. It is to be regretted that some of the members of the Committee are not also members of the Canadian Institute, but it is hoped, and believed, that members of the Institute will render a full and constructive discussion of all the features of the Specification.

F. E. TURNEAURE,* M. AM. SOC. C. E. (by letter).†—Since both live loading and impact have been recent subjects discussed by the American Railway Engineering Association, the work of the Committee was greatly simplified, and in general consisted of a careful review of the publications of that Association and the consideration of new material or new aspects that may have appeared since that date.

Engine Loading.—A detailed study of the relative effects of various engine loadings in use and the standard Cooper series has been published by the American Railway Engineering Association.‡ The most significant of these diagrams are re-published as Figs. 10 to 15, inclusive, and show clearly the relative effects of the Cooper series as compared with engine loadings in use. Of these diagrams, Fig. 10, Group 1, shows seven heavy road locomotives of various types; Fig. 11, Group 2, shows six locomotives, representing the heaviest of the various types of engines now generally used in special service; Figs. 12 and 13 show the actual moments and shears produced by the loading shown in Figs. 10 and 11, and, likewise, the moments and shears produced by Cooper's E-60 loading; and Figs. 14 and 15 show the moment and shear curves for the locomotives covered by Groups 1 and 2 (Figs. 10 and 11), plotted as percentages of Cooper's E-60 loading.

As a basis of this study, the Cooper series was recommended by the Committee of the American Railway Engineering Association and adopted by that Association. The reasons presented by the Committee were as follows:

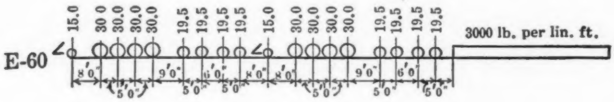
1.—No one existing type of locomotive loading gives maximum moments and shears for spans of all lengths, whereas the system recommended does approximate the high points of the curves of all the existing types.

2.—This system of loading produces stresses slightly smaller in short-span bridges and slightly greater in long-span bridges than those produced by the heaviest types in operation, which adequately provides for the engines now in use, and for future development in engine and car loadings which will increase the load per foot on long structures.

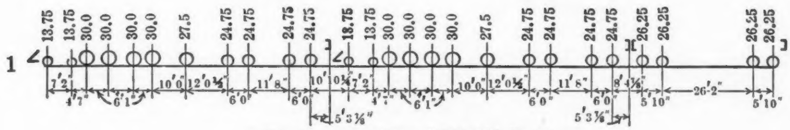
* Madison, Wis.

† Received by the Secretary, November 26th, 1921.

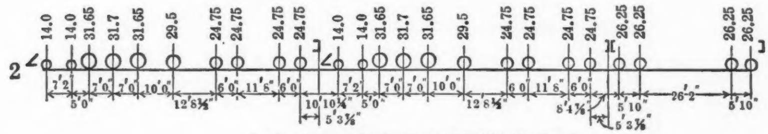
‡ *Proceedings*, Am. Ry. Eng. Assoc., Vol. 21, p. 561.



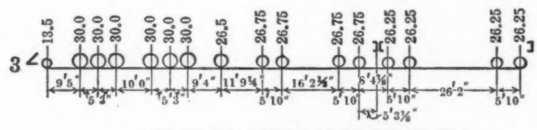
Cooper's E-60



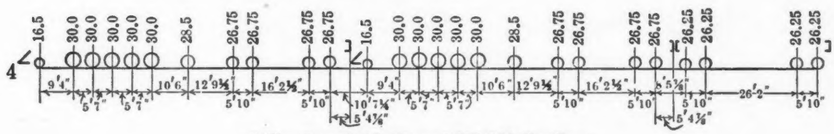
2 Engines 4-8-2B and 210,000-lb. Cars



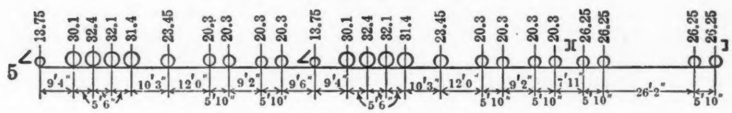
2 Engines 4-6-2B and 210,000-lb. Cars



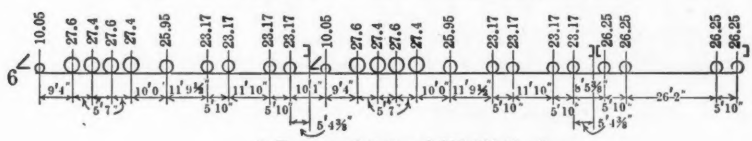
1 Engine 2-6-6-2B and 210,000-lb. Cars



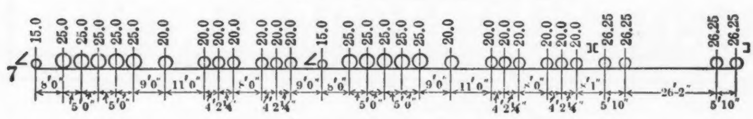
2 Engines 2-10-2B and 210,000-lb. Cars



2 N. Y. C. R. R. Engines H-7A 2-8-2 and 210,000-lb. Cars



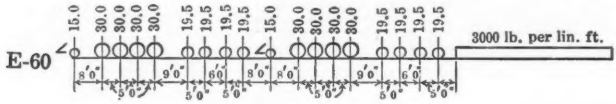
2 Engines 2-8-2 and 210,000-lb. Cars



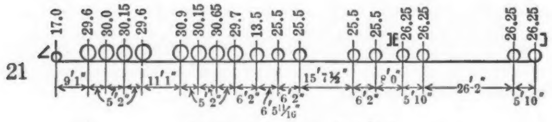
2 Sante Fe Type Engines 2-10-2 and 210,000-lb. Cars

Note: Engines 1 to 7, inclusive, are followed by trains of 210,000-lb. cars.
Concentrations are for 1 rail, in thousands of pounds.

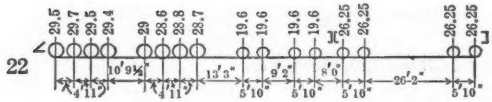
FIG. 10.



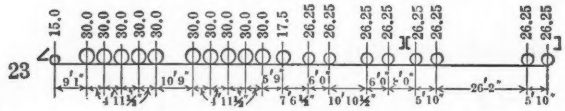
Cooper's E-60



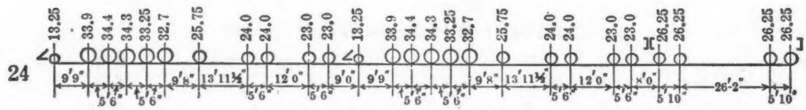
1 Virginian Ry. 2-8-8-2



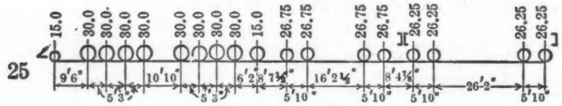
1 L.S. & M.S. Mallet 0-8-8-0



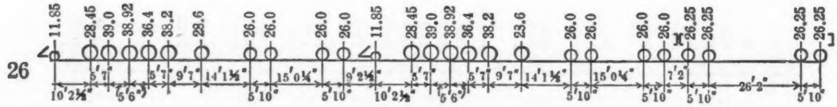
1 Virginian Ry. 2-10-10-2



2 Erie Engines 2-10-2



1 Engine 2-8-8-2B



2 Pennsylvania R. R. NIS Engines 2-10-2

Note: Engines 21 to 26, inclusive, are followed by trains of 210,000-lb. cars.
Concentrations are for 1 rail, in thousands of pounds.

FIG. 11.

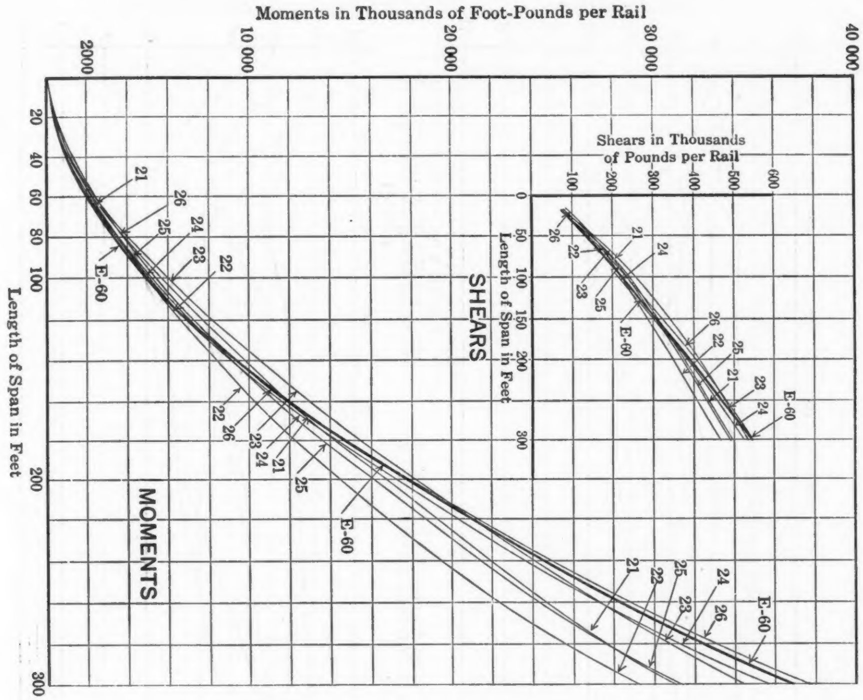

Moments in Thousands of Foot-Pounds per Rail
Length of Span in Feet
MOMENTS
SHEARS
Shears in Thousands of Pounds per Rail
Length of Span in Feet
E-60
21
22
23
24
25
26

Fig. 12.

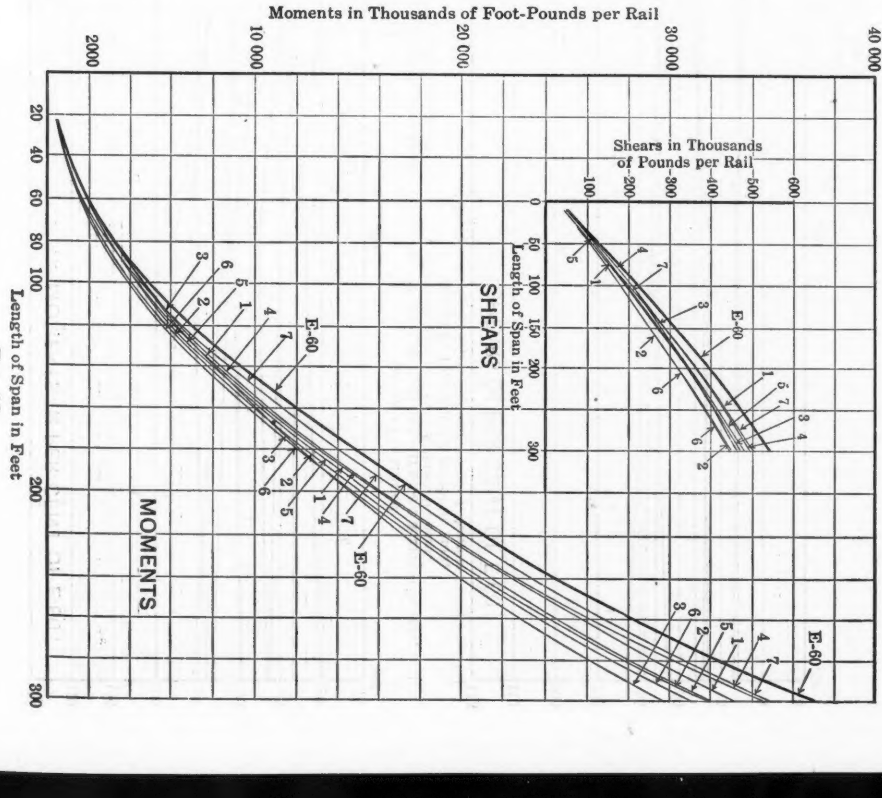

Moments in Thousands of Foot-Pounds per Rail
Length of Span in Feet
MOMENTS
SHEARS
Shears in Thousands of Pounds per Rail
Length of Span in Feet
E-60
1
2
3
4
5
6
7

Fig. 13.

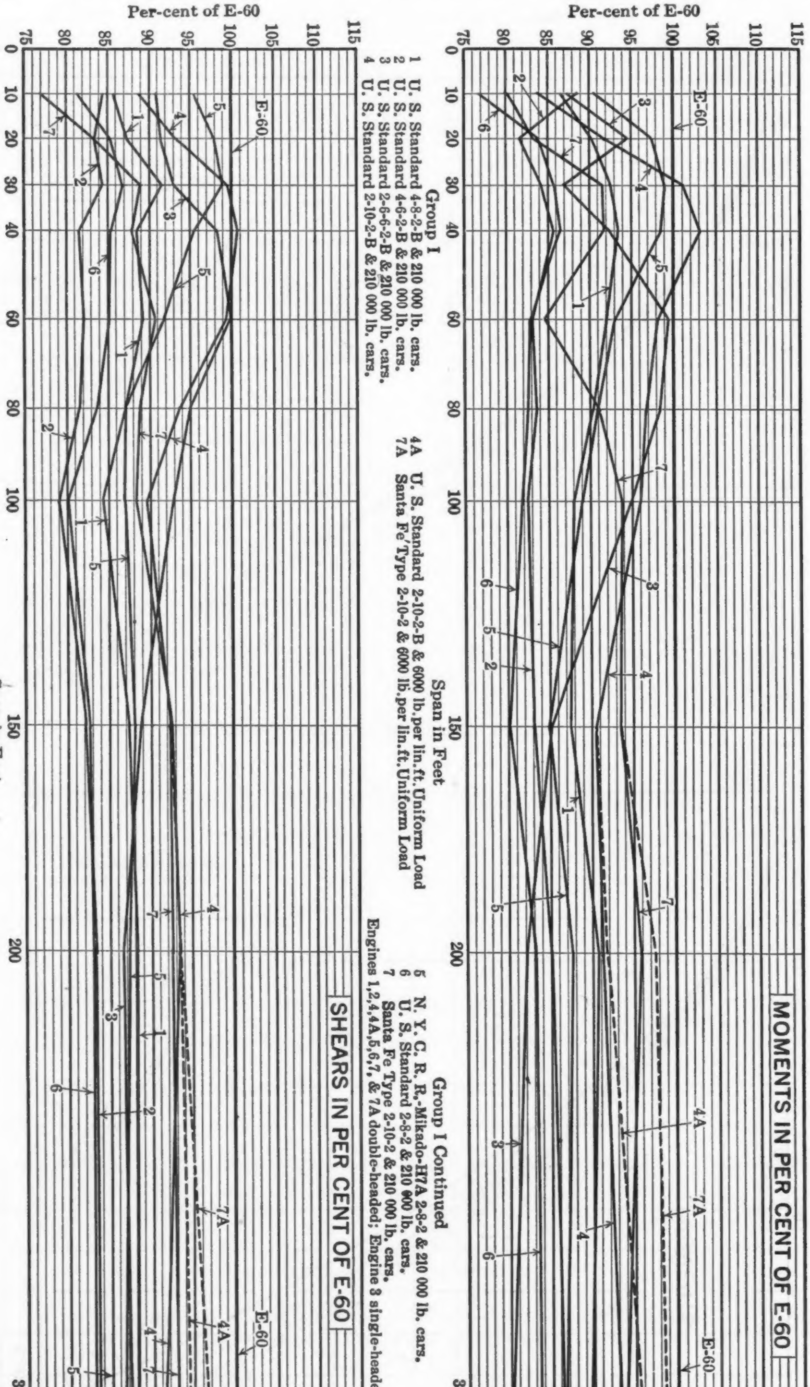
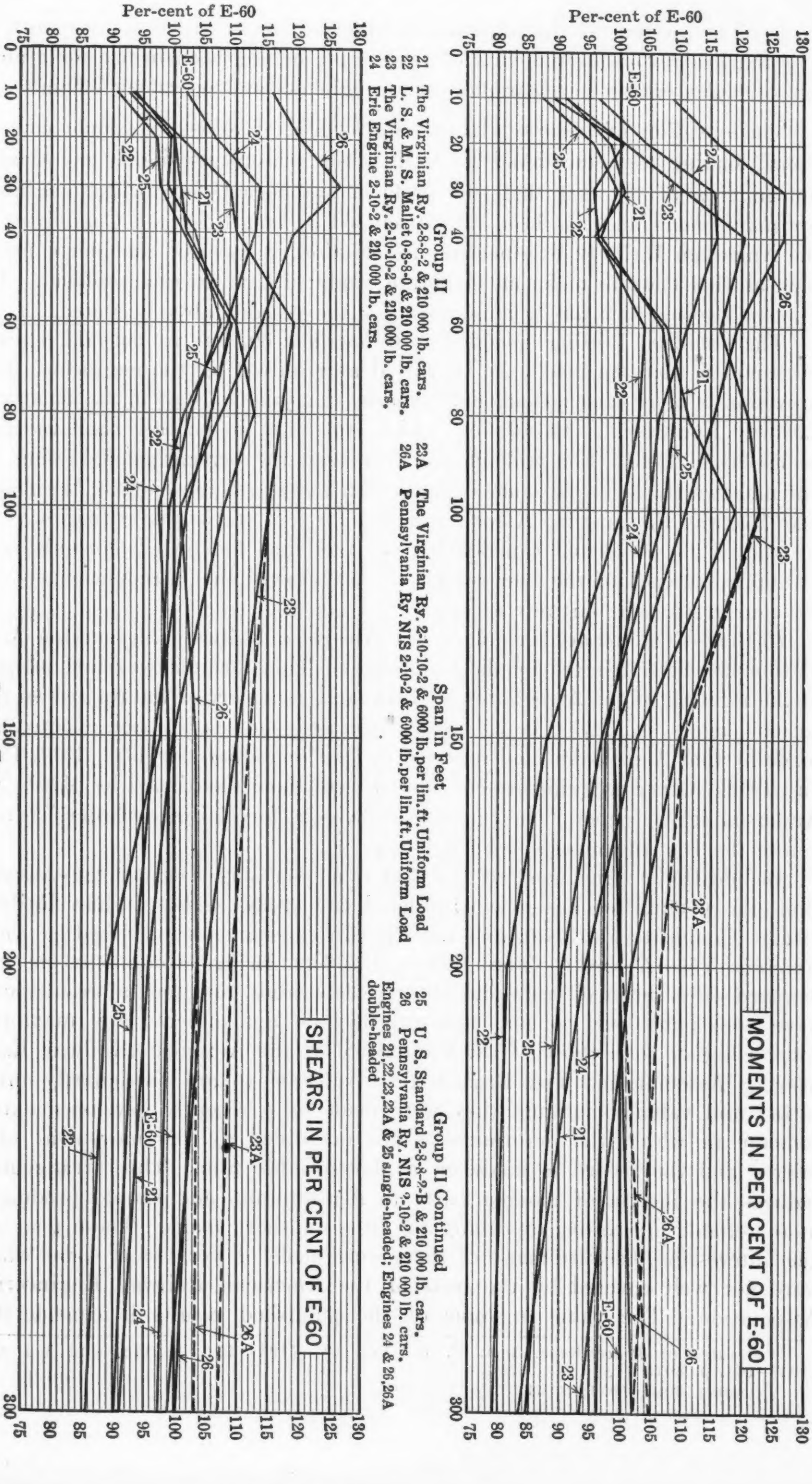

MOMENTS IN PER CENT OF E-60
SHEARS IN PER CENT OF E-60
Per-cent of E-60
Span in Feet
Group I
1 U. S. Standard 4-8-2-B & 210 000 lb. cars.
2 U. S. Standard 4-6-2-B & 210 000 lb. cars.
3 U. S. Standard 2-6-6-2-B & 210 000 lb. cars.
4 U. S. Standard 2-10-2-B & 210 000 lb. cars.
4A U. S. Standard 2-10-2-B & 6000 lb. per lin. ft. Uniform Load
7A Santa Fe Type 2-10-2 & 6000 lb. per lin. ft. Uniform Load
Group I Continued
5 N. Y. C. R. R.-Mikado-H7A 2-8-2 & 210 000 lb. cars.
6 U. S. Standard 2-8-2 & 210 000 lb. cars.
7 Santa Fe Type 2-10-2 & 210 000 lb. cars.
Engines 1,2,4,4A,5,6,7, & 7A double-headed; Engine 3 single-headed

Fig. 14.



Group II

21 The Virginian Ry. 2-8-8-2 & 210 000 lb. cars.
22 L. S. & M. S. Mallet 0-8-8-0 & 210 000 lb. cars.
23 The Virginian Ry. 2-10-10-2 & 210 000 lb. cars.
24 Erie Engine 2-10-2 & 210 000 lb. cars.
23A The Virginian Ry. 2-10-10-2 & 6000 lb. per lin. ft. Uniform Load
26A Pennsylvania Ry. NIS 2-10-2 & 6000 lb. per lin. ft. Uniform Load

Group II Continued

25 U. S. Standard 2-8-8-2-B & 210 000 lb. cars.
26 Pennsylvania Ry. NIS 2-10-2 & 210 000 lb. cars.
Engines 21, 22, 23, 23A & 25 single-headed; Engines 24 & 26, 26A double-headed

Span in Feet
FIG. 15.

3.—This system is the adopted standard of measurement of strength of existing bridges on the majority of roads, and, having been in use for a number of years, conveys a clear picture to engineers and operating officials.

The question of some equivalent uniform load or other simple conventional loading was considered, as well as the reasons previously mentioned; but in view of the fact that this problem has been before bridge engineers for many years and has been also discussed at length by the Committee of the American Railway Engineering Association, it was not considered necessary at this time to make an elaborate study of this general problem. The Cooper loading therefore was adopted for the reasons already stated.

The standard loading specified is Cooper's E-60, but for special cases it is provided that the load may be varied as required by the engineer. This provision is more elastic than that given by the specifications of the American Railway Engineering Association, which state that in no case shall the load be less than E-45. For multiple-track bridges, the percentage reduction for one and two tracks is the same as that given in the specifications of the American Railway Engineering Association; but for four tracks, the reduction is 30, instead of 25, per cent. Considering the fact that the impact formula is to be applied to each of the four tracks, it was believed that a reduction of 30% for four tracks was entirely safe.

Impact.—The impact formula of the American Railway Engineering Association has been adopted by the Committee. This formula was first adopted at the meeting of the Association in 1918, and, as a part of the revised specifications, again in 1920. It was based primarily on the impact experiments made by that Association and reported in its *Proceedings* for 1911, 1914, 1917, and 1918. Other experiments considered were those mentioned by Mr. C. W. Anderson in his paper, "On Impact Coefficients for Railway Girders",* read before the Institution of Civil Engineers.

In addition to the work of the American Railway Engineering Association, the Committee has also considered the study of the Indian Railway Bridge Committee, 1919-20, and the English experiments published in *Engineering News-Record.*† The Indian Railway Bridge Committee is now engaged on a series of tests and theoretical studies, and partial results only have been published; so far, however, they do not indicate any maximum results higher than those of the Committee of the American Railway Engineering Association. The English tests referred to are particularly interesting and valuable because they were made by a recently developed extensometer in which the deformations are magnified by the movement of a mirror and the record is made on a photographic film. This arrangement reduces the inertia of moving parts to a minimum, and should produce a more reliable apparatus, especially for tests on short spans. These tests are also interesting, because many of them were made on very short spans which were not well covered by the tests of the American Railway Engineering Association. The value of some of the published results is considerably

* *Minutes of Proceedings*, Inst. C. E., Vol. CC (1914–15), p. 178, and Vol. CCI (1915–16), p. 301.

† October 20th, 1921, p. 642.

affected by reason of the shifting of the zero of the diagram during the test, and because of the great differences between the calculated static stresses and the observed slow speed values. Comparing these experiments with those of the American Railway Engineering Association, it appears that results for spans of more than 40 ft. are fairly well confirmed by the English tests. On the whole, the English tests show smaller results for these spans than those of the American Railway Engineering Association. For spans less than 30 or 40 ft., the results obtained by the Committee of the American Railway Engineering Association were not generally satisfactory, on account of instrumental vibrations, and in the report of that Committee not much weight was given to results on such short spans. In selecting a formula, the Committee of the American Railway Engineering Association was guided primarily by the results of tests of spans of 30 ft. and more, and for shorter spans by past practice, in which the impact has been taken at about 100 per cent. These considerations resulted in the formula adopted, which gives a reversed curve, starting at 100% for zero span and dropping to 75% for 100-ft. spans. These values are the same as those given by the old formula of the American Railway Engineering Association, and between these span lengths the new formula gives somewhat higher values. For spans exceeding 100 ft., the new formula gives values less than the old one, the difference gradually increasing with the span length.

In discussing the type of formula to be preferred, two main points were considered: First, the proper impact values for short spans, say, up to 25 or 30 ft.; and, second, the proper values for spans longer than about 150 ft. For intermediate lengths, there is little difference between the old and new formulas, and no material change was suggested by any member of the Committee.

For very short spans, the principal results are from the English tests mentioned, which include spans as short as 7½ ft. Two formulas have been proposed by those in charge of these tests, the one preferred being:

$$I = \frac{120}{90 + L}$$

This formula gives 133% impact at zero span and somewhat lower values than the American Railway Engineering Association's curve between spans of 35 and 200 ft., and higher values for longer spans.

Another impact curve which gives a value exceeding 100% for zero span, and which conforms well to the test of the American Railway Engineering Association, is that of Henry B. Seaman, M. Am. Soc. C. E.* This formula is an elliptical curve, tangent to the Y-axis at zero span and to the X-axis at 1 000 ft. span.

Another formula of the same type as the old formula of the American Railway Engineering Association was suggested by a member of the Committee, namely,

$$I = \frac{175}{140 + L}$$

This formula gave an impact of 125% for zero span.

* *Transactions*, Am. Soc. C. E., Vol. LXXV (1912), p. 318.

For spans of 200 ft. and more, all experimental results indicate that the old curve of the American Railway Engineering Association is much too high, but naturally there is considerable difference of opinion as to the proper values to adopt.

The desirable type of formula appeared to be dependent on whether or not the impact percentage for zero span was to be made more than 100 per cent. If increased to 125 or 130%, the old type of formula could be made to fit the tests satisfactorily; but if 100% was taken as the value for zero span, then such a curve, if kept high enough for spans of less than 150 ft., would be too high for long spans. After considerable discussion, it was decided to adopt the new formula of the American Railway Engineering Association, making the impact 100% for zero span.

It was felt that although it is probable that a somewhat larger impact actually occurs for very short spans on rigid supports, the long experience of engineers in designing and maintaining short-span structures designed on the basis of 100% impact for zero span and operated under very large overloads, should be given much weight. Such experience indicates that the value of 100% is sufficiently high to insure safe and durable structures under the high stresses allowed by the specifications of the American Railway Engineering Association for the rating of existing structures. The relative safety and durability of short-span bridges is probably due, in part, to the plate-girder and I-beam type used for such spans, and the Committee of the American Railway Engineering Association gave some consideration to the question of whether a different base unit stress should not be allowed for such structures. It was concluded, however, to adhere to the same base unit stress and to adopt 100% for the impact ratio for zero span. These same considerations were reviewed by the Committee, with the result that the majority of the members favored the formula given in the new specifications of the American Railway Engineering Association.

BURTON R. LEFFLER,* M. AM. SOC. C. E. (by letter)†.—The Committee has presented three column formulas. It was impossible for the members of the Committee to agree on one formula. It was thought that any one of the formulas was in accord with good practice.

The straight-line formula and the parabolic formula give practically the same average unit stresses. The Rankine formula gives different average stresses. The writer prefers the parabolic formula if it is necessary to consider a column from a purely empirical standpoint. The parabolic curve fits the tests through the whole range up to the point of tangency with Euler's curve. Neither of the other two formulas has this quality; it is necessary to state one or more limits. The writer believes that it is a mistake to consider column tests from a purely empirical standpoint.

It has become the practice to draw a line through plotted results and assume that the equation of the line represents the whole phenomena.

* Cleveland, Ohio.

† Received by the Secretary, November 29th, 1921.

Euler's formula fits the tests for long columns, say, for $\frac{l}{r}$ more than 150, when due consideration is given to the degree of fixation at the ends. A long thin column, such as a sword blade or a steel straight edge, may fail by pure buckling, with unit stresses below the elastic limit. Euler's formula expresses the law of failure for a straight spring.

A short column fails because the unit stress at some section exceeds the strength, or the elastic limit, of the material. It does not fail because the average unit stress is high. For very short columns the average unit stress is equal to the highest unit stress. At any section of a column there is an axial force and a bending moment. For short columns the bending moment is zero, or nearly zero, depending on how closely the column is centrally loaded.

The problem of the columns is one of bending moment. The very fact that columns fail with an average unit stress below the strength of the material is evidence that it is impossible to centralize the load at every section, regardless of how central the load is at the ends of the columns. This is due to the non-homogeneous character of the material, caused by imperfection of manufacture and fabrication. For like reasons a correct theory of the failure of a tension member includes the consideration of a bending moment. Every structural member in a bridge has a bending moment; in tension members its effect is negligible, while in columns it is important.

At any section of a column the bending moment may be assumed as caused by the load being eccentrically applied. There may be an additional eccentricity due to the load being visibly non-central at the ends of the column.

As soon as an eccentricity is assumed, a correct column formula can be developed. The formula is:

$$p = \frac{f}{1 + \frac{e\,c}{r^2} \sec \frac{l}{2\,r} \sqrt{\frac{P}{E}}}$$

which is known as the secant formula, and is somewhat cumbersome to apply. p is the average unit stress, e the eccentricity, r the radius of gyration, l the length of the column, and c the distance from the gravity axis to the extreme fiber. The other terms are well known. The formula is for columns having knife edges (not pins) at the ends. For other end conditions, suitable values of l must be used. The reader is referred to the different treatises on mechanics of materials.

All the terms are determinate for the ideal column; but for the practical column, visibly centrally loaded, e is unknown; it must be determined by experiments. The results of the experiments must be interpreted, keeping in mind the relations of the other terms of the secant formula.

The second term in the denominator of the formula determines that part of f caused by the bending moment. It has little resemblance to the apparently equivalent terms in the formulas proposed by the Committee.

A column formula which represents the bending fiber stress without the term, c, must be wrong. The distance to the extreme fiber and the radius of gyration are fundamental terms in any formula which purports to give a bending fiber stress.

The purely empirical formulas proposed by the Committee make no difference between an I-section and a box-like section. According to these formulas, if $\frac{l}{r}$ is the same, the sections are of equal value.* Contrast the fine instinct of the late Sir Benjamin Baker, Hon. M. Am. Soc. C. E., in using the circular section for the Firth of Forth Bridge, with the blindness exhibited in a collapsed American bridge.

The secant formula compels the designer to consider the section of a column in a thorough manner. It is interesting to note that Tredgold and Gordon considered the c term to be important in a column section. Determining the area of a column section should be something more than a blind use of $\frac{l}{r}$ in a formula.

The writer regards the tests of columns as useless until engineers rationalize the results. The form of the section and the difference between a purely rolled column and a built-up column should be given more study.

Where a problem admits of a partial rational treatment, it seems wrong to solve it in a purely empirical way. Engineers should couple rationalizing and experimenting.

Let us consider further the formulas: Take $\frac{l}{r} = 40$. The fiber stress due to bending is 2 000 lb. for the straight-line formula, 400 lb. for the parabolic formula, and 1 700 lb. for the Rankine formula. Now, as the straight-line and parabola almost coincide, why the difference? The writer will leave this to the reader to think about. If the shear for column lacing is calculated, marked differences will also be found.

The secant formula includes Euler's formula. Arthur Morley deduces the formula by a substitution of terms in the chain of reasoning leading to Euler's formula†.

The secant formula fits the results of tests closer than any other formula.‡

The advocates of the Rankine formula claim that it is rational, but how can it be with the c term omitted? The late J. B. Johnson, M. Am. Soc. C. E., deduced the formula§:

$$p = \frac{f}{1 + \frac{e\,c}{r^2} + \frac{f - p}{10\,E}\left(\frac{l}{r}\right)^2}$$

The formula is based on the bending diagram as being a parabola; for the secant formula, the diagram is a sinusoid. Rankine's formula is derived

* "Modern Framed Structures", Pt. 3, Article 55.

† Morley's "Strength of Materials", Article 104.

‡ "Modern Framed Structures", Pt. 3, Article 43.

§ "Modern Framed Structures", Article 134 of Fourth Edition.

from Johnson's formula by omitting the second term of the denominator and substituting an empirical constant for the coefficient of the last term. Such transformation is not valid; it shows Rankine's formula to be strictly empirical. On account of this transformation, the formula does not fit test results as well as the parabolic formula.

A riveted column should be designed for a section less than the gross. Riveted tension members are designed for net section. Probably a somewhat less allowance would do for columns. Riveting should be reduced to a minimum, and it should be symmetrical at any section of the column. The ordinary column composed of two channels connected by single lacing is unsymmetrically riveted.

Symmetry should be a cardinal principle in column design, it reduces the unknown eccentricity; minimum fabrication is also conducive to this end.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

ODORS AND THEIR TRAVEL HABITS

Discussion*

By Messrs. Paul Hansen, George C. Whipple, Stephen DeM. Gage, I. S. Osborn, Rudolph Hering, Olin H. Landreth, Andrew J. Provost, Jr., Robert Spurr Weston, Alexander Potter, and Caleb Mills Saville.

Paul Hansen,† M. Am. Soc. C. E. (by letter).‡—The author deserves great credit for preparing a paper on the elusive subject of odors. As he has pointed out, it is a subject which is so difficult to treat in a tangible and scientific manner that engineers have hesitated to make it the subject of discussion before technical societies. It would seem, however, that a fuller discussion of the subject, on the part of engineers and chemists, might lead to more standardized, definite, and scientific means for measuring the quality, intensity, and carrying powers of odors, than are in use at the present time.

As the author invites experience from other engineers, it might not be inappropriate to refer to a report on odors from an oil refinery, recently prepared by the writer in collaboration with F. C. Dugan, Assoc. M. Am. Soc. C. E., Chief Engineer of the State Department of Health of Kentucky, and G. C. Smith, Professor of Engineering Chemistry at the University of Cincinnati. This investigation seemed to call for some general remarks on the subject of odors, which are quoted essentially as follows:

"The effect, psychology, and significance of odors constitute a rather confusing subject especially to the lay mind and frequently to the legal mind. A primitive sense of odor was apparently designed by Nature to (1) distinguish inedible from edible substances; (2) as a means of recognizing things and creatures; and (3), in some instances, as a warning against danger. Generally speaking, the disagreeable odors were those that indicated inedible substances and dangers. Persons instinctively associate disagreeable odors with dangers to health. So strong are these instinctive feelings that it is difficult to get people to believe that their health and lives are not in danger when in the presence of strange or disagreeable odors.

"As a matter of fact it can be very clearly demonstrated that odors in and of themselves are not dangerous to health, although they may indicate the

* This discussion (of the paper by Louis L. Tribus, M. Am. Soc. C. E., published in August, 1921, *Proceedings*, and presented at the meeting of November 16th, 1921), is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion.

† Chicago, Ill.

‡ Received by the Secretary, September 6th, 1921.

presence of substances which if ingested or inhaled in concentrated form may have physiological results.

"In addition to natural odors, man's ingenuity has introduced many industrial processes that have odors. Most of these industrial odors when first encountered are regarded as disagreeable in varying degrees, certainly they are nearly always objected to when introduced near places of abode, not so much because they are hurtful, as because they prevent the free enjoyment of the atmosphere as Nature made it. Where industrial odors are accepted as a necessary evil, as is apt to be the case in a greater or lesser degree in all large cities and industrial communities, people become inured to the odors and either do not notice them or, in some instances, actually develop a liking for them. Grain mills, woolen mills, cotton mills, dye-works, soap factories, slaughter-houses, sugar refineries, shoe factories, paper mills, rendering plants, tanneries, etc., all have characteristic odors, often mild, but sometimes rather strong, to which whole communities become accustomed and accept without complaint. There are certain industries, however, that produce odors so strong and so unpleasant that they are rarely permitted in residence communities. Among these 'offensive industries' are rendering plants, slaughter-houses, certain chemical processes, glue works, etc.; but even some of these odors are tolerated and accepted as a matter of course and as unavoidable. Perhaps the most notable example of this is the notorious stockyards' odor that frequently pervades the whole South Side of the City of Chicago, in which are included many fine residence districts. Aprôpos of this, the so-called inquiring reporter of the *Chicago Tribune* one day made inquiry at random of ten people on the South Side of Chicago regarding their attitude toward the odors from the stockyards. Only one stated that the odors were disagreeable, nine were indifferent to the odors, and one admitted that to him the odors were actually agreeable. In short, it is difficult to decide when an odor may be regarded as a public nuisance and when it may not be so regarded; the most one can say is that an odor is a nuisance if the public is not used to it and does not want it in the neighborhood. The same odor may not be a nuisance if the public is used to it or is willing to tolerate it for economic reasons or otherwise.

"Odors from oil refineries seem to fall in a middle class. Though some very sensitive persons may be nauseated by oily odors, the average person is not apt to describe them as distinctly disagreeable in concentrations in which they would ordinarily be found even in close proximity to a refinery. Nevertheless, the same average person would be very much averse to having these odors in and about his residence, much from the same point of view as people nowadays avoid living on streets extensively traveled by automobiles, or regard the odor from recently oiled streets as objectionable in their homes.

"In a broad way, the principle seems to be fairly well established in practice (and, apparently, in law as well), that essential industries even though odor-producing must be accorded an existence when once established. Recently, there has grown a tendency to regulate in advance the location of the more objectionable odor-producing industries by means of zoning ordinances or other regulatory devices, which accord suitable space to all industries, yet protect the property holders and stabilize real estate value.

"Some industries produce corrosive, blighting, and even dangerous gases, but these effects must be considered as distinct from odors, because certain of these gases have little if any odor. Zinc-refining and acid-making industries fall within this class. An oil refinery, however, could hardly be classed as objectionable on these grounds, as the concentration of escaping vapors is never so great as to affect vegetation or injure the respiratory tract. This is evidenced by the general freedom of employees of oil refineries from afflictions of the respiratory organs. Refinery vapors may contain certain substances that have a corrosive effect on metals and which may affect certain kinds of

paints. These effects are most noticeable when the oil is rich in sulphur compounds. We are unable at the present time to make any more precise statement regarding the nature of these substances, but certain observed effects on paint, especially on the office building at the plant, indicate the presence of small quantities of some substance or substances capable of injuring lead paint. The testimony indicates that at rare intervals the paint on houses in the city is affected somewhat by these gases. Probably, most of the objectionable compounds come from the tail-house where unfinished oils are exposed and also from the agitator where gases containing sulphur compounds may escape in considerable volume."

In connection with the investigation and the report from which this quotation is taken, a limited number of observations were made on the effect of wind, temperature, and barometric condition on the dissemination of odors, in which the records of the United States Weather Bureau were found to be of great assistance and value. Owing to the limitations under which the investigation was made, however, these observations were not sufficiently long continued or sufficiently well carried out to warrant giving them any space in this discussion. However, they would suggest the possibility that it may be practicable to obtain a classification of odors according to some generally recognized standards. It may also be possible to trace the dissemination of odors with parallel observations on meteorological and other conditions, so that it may be feasible after a sufficiently large mass of data has been accumulated to predict, under any given set of conditions, whether or not a sewage plant or a so-called offensive industry is likely to become the subject of complaint.

In regard to the author's remarks on the feasibility of preventing odors rather than having them abated, the writer wishes to emphasize the value of zoning ordinances in this connection. Zoning ordinances are now authorized in a number of States, and they constitute a most excellent means whereby odor-producing industries can be located in advance so that they will not prove to be a discomfort or injurious to property values.

GEORGE C. WHIPPLE,* M. AM. SOC. C. E.—The author has done well to bring to the attention of engineers this vexing question of odors and their travel habits. This question is inseparably connected with sewage disposal, stream pollution, garbage disposal, and many chemical industries. Odors that are offensive to human beings and travel over wide areas are public nuisances and come within the scope of police power, a power exercised in this case by State or local boards of health. There is hardly a question in public health administration which is more troublesome than this matter of odors. Several things contribute to make just decisions relating to odor nuisances difficult: The question of whether bad odors affect health; differences in people as to their sensibilities to odors, which are not only individual, but sometimes racial; physiological problems which are not well understood; the effects of temperature, moisture, and air movement, meteorological matters which are directly concerned with the dissemination of odors; engineering problems of odor control by methods of prevention or by air purification; the problem of comparing the cost of such protective measures with the damage inflicted,

* Cambridge, Mass.

a matter of economics; and the question of the appropriateness of odors to certain places—a psychological question. These factors are often so mixed and tangled that it is no wonder that the legal and administrative phases of the subject are difficult. Perhaps, the solution of particular problems may be made easier if these different elements of the subject are considered separately under the following heads: (1) Physiology of odors; (2) relation between odors and health; (3) meteorological factors of odor travel; (4) governmental control of odors; and (5) prevention of odors.

The Physiology of Odors.—The speaker has recently had the pleasure of looking over the manuscript of a forthcoming book on "Smell, Taste, and Allied Senses in the Vertebrates", by Professor George H. Parker, of Harvard University, in which a chapter is devoted to the anatomy of the olfactory organs and another on the physiology of olfaction.

There has been some controversy among physiologists as to the mechanism by which the so-called sense of smell is exercised; but it is now recognized that two classes of substances produce nasal stimulation, namely, irritants, which act on the free ends of the trigeminal nerve, and true odors, which affect the terminal cells of the olfactory nerves. This is an important distinction. Some substances, like sulphurous acid, are irritants and have no true odor; others, like the oil of vanilla, have a true odor, but are not irritating; but some substances, like tobacco smoke, affect both sets of nerves, although this may be due to the mixed character of the smoke.

It appears to be well established that both odors and nasal irritations are caused by actual substances carried by the air into the nasal chamber. The quantity of these substances required to produce sensation is almost infinitely small. The ends of the nerve cells are not uniformly distributed over the nasal passages, and in ordinary breathing the air does not come in contact with them. Sniffing, however, brings the air into those parts where the nerves are located. Diffusion of the air in the nose may carry small amounts of the odoriferous or irritating substances to the nerve cells, and, once stimulated, involuntary sniffing may occur. The air may reach the nerve cells through the nose during inspiration, as in the ordinary process of smelling, or the odors may be carried to the cells from the mouth during respiration, as in tasting food. Many so-called tastes are really odors; holding the nose prevents respiration through it and thus prevents the taste (odor) of castor oil, for instance, from being noticed.

The physical state of the minute particles which produce irritation or odor does not appear to be known. Possibly they are electrically charged. It is certain that they affect the nerves most when accompanied by moisture in the air. "They do not emanate from their source and disperse in all directions as sound and light waves do." They result from evaporation or volatilization and are carried by the air as it moves, being spread through it by processes of mixing.

In the nose the olfactory membrane is covered by mucus into which the olfactory hairs project and beyond which they do not extend. The mucus apparently dissolves the odoriferous particles which thus come into intimate contact with the olfactory hairs.

The quantities of substances required to stimulate the trigeminal and olfactory nerve cells are extraordinarily minute. Many years ago, when studying the odors produced by microscopic organisms in water, the speaker found that oil of peppermint could be recognized when diluted with 50 000 000 times its volume of water, and that the essential oils found in several of the algæ could be detected in dilutions of from 5 000 000 to 25 000 000. Various devices have been used recently by physiologists in studying olfactory acuity. Some of these olfactometers are most ingenious, especially that of Allison and Katz.*

As a result of various tests, it has been found that certain odors can be recognized in the quantities given in Table 1. Different observers give somewhat different values, as might be expected.

TABLE 1.—QUANTITIES OF SUBSTANCES REQUIRED FOR MINIMUM STIMULATION.

Substance.	Thousandths of a milligramme per liter of air.
Camphor	5
Ether	1
Sulphureted hydrogen	0.5
Citral	0.5 to 0.1
Heliotropin	0.1 to 0.005
Cumarin	0.05 to 0.01
Vanillin	0.005 to 0.0005
Chlorphenol	0.004
Oil of rose	0.0005
Mercaptan (garlic odor)	0.00004
Artificial musk	0.00004

Messrs. Allison and Katz† has given a table, part of which is reproduced herein as Table 2.

TABLE 2.—CONCENTRATIONS OF MILLIGRAMMES OF CHEMICAL PER LITER OF AIR.

Chemical.	Intensity of Odor.				
	Detectable.	Faint.	Noticeable.	Strong.	Very strong.
Ethyl ether	5.833	10.167	14.933	17.667	60.600
Chloroform	3.300	6.800	12.733	28.833	46.666
Ethyl acetate	0.686	1.224	2.219	4.457	6.733
Ethyl mercaptan	0.046	0.088	0.186	0.357	0.501
Pyridine	0.032	0.146	0.301	2.265	5.710
Oil of peppermint	0.024	0.032	0.109	0.332	0.348
Iodoform	0.018	.....	.....	.....	.....
Methyl isothiocyanate	0.015	0.039	0.067	0.108	0.144
Butyric acid	0.009	0.021	0.066	0.329	0.580
Allyl isothiocyanate	0.008	0.012	0.024	0.030	0.201
Propye mercaptan	0.006	0.020	0.028	0.043	0.054
Amyl thioether	0.001	0.007	0.011	0.012	0.015
Artificial musk	0.00004	.....	.....	.....	.....

* "An Investigation of Stenches and Odors for Industrial Purposes", *Journal*, Industrial Eng. Chemists, Vol. XI (1919), p. 336.

† *Journal*, Industrial Eng. Chemists, Vol. XI (1919), p. 338.

It would appear from these data that the olfactory nerves are more sensitive than those which give rise to nasal irritation, but it is probably unsafe to state this as a general truth.

In his studies of the odors of organisms in water, the speaker has observed that as odors decreased in intensity they changed in quality. This is probably due to the fact that few odors are pure, that is, due to a particular substance, most odors are mixtures. When these mixtures are diluted, the less sensitive constituents lose their effect first, while the more sensitive ones persist; the result being a change of odor. This is also true in the case of substances which produce both irritation and odor, the odor may persist after the irritation has ceased, or when increasing in intensity the odor is noticed first and this is followed by irritation. Tobacco smoke is an example. There may be mixtures of substances which act chemically on each other, so that the result is actually a different odor, and not merely a mixture of odors. On a more intense scale, it is well known that one odor may mask another and that a strong agreeable smell may be substituted for an unpleasant one. The little known subject of ratios of odors underlies the practice of "deodorization".

Temperature has an important influence on olfactory acuity, an influence partly physical and partly physiological. Heat increases the evaporation and volatilization of most odoriferous substances. Cold air drawn into the nose chills the mucous lining and perhaps retards the concentration of the particles in it. Conversely, cold fresh air makes the scent more keen. Cold water drawn into the nose greatly reduces the power of detecting odor.

Another important physiological phenomenon is that of fatigue and exhaustion of the olfactory organs. The continual smelling of an odor exhausts the power to recognize it. This may be a matter of a few minutes, or it may be longer. Every one knows the phenomenon of "getting used to an odor". Every one knows that breathing fresh air removes the last traces of odors and restores the power to recognize those which are very faint. An ocean voyage does this to a marked degree. On approaching the Japanese coast and while still several miles from it, the speaker will never forget how, after ten days on the Pacific, he enjoyed the "land smell", the smell of the "good brown earth".

The quality of an odor is usually designated by reference to the substance with which it is associated. It is different from taste, of which there are four clearly marked qualities: sweet, sour, salt, and bitter. Odor designations are numberless and constantly changing, but attempts have been made to classify them. Haller's classification has nine groups, as follows:

1.—Etherial odors: odors of fruits, beeswax, ethers; three subdivisions.
2.—Aromatic odors: odors of camphor, cloves, lavender, lemon, bitter almond; five subdivisions.
3.—Balsamic odors: odors of flowers, violet, vanilla, cumarin; three subdivisions.
4.—Ambrosial odors: odors of amber, musk; two subdivisions.
5.—Alliaceous odors: odors of hydrogen sulphide, hydrogen arsenide, chlorine; three subdivisions.
6.—Empyreumatic odors: odors of roast coffee, benzole; two subdivisions.
7.—Caprylic odors: odors of cheese, rancid fat; two subdivisions.
8.—Repulsive odors: odors of deadly nightshade, bedbug; two subdivisions.
9.—Nauseating odors: odors of carrion, feces; two subdivisions.

Another classification (Henning's) has six groups, as follows:

1.—Spicy odors, such as those of fennel, sassafras oil, anice, and cloves.
2.—Flowery odors, such as those of heliotrope, cumarin, and geranium oil.
3.—Fruity odors, such as those of oil of orange, citronella, oil of bergamot, and acetic ether.
4.—Resinous or balsamic odors, such as those of turpentine, Canada balsam, and eucalyptus oil.
5.—Burnt odors, such as those of tar and pyridin.
6.—Foul odors, such as those of carbon bisulphide and hydrogen sulphide.

All these groups are artificial and unsatisfactory. They have a psychological rather than a physiological basis.

Relation Between Odors and Health.—Do odors affect health? This question cannot be properly discussed without drawing a distinction between true odors and irritants and without a clear understanding of what is meant by "health".

In a physiological sense, meaning a condition of the body in which it operates normally, the effect of odors on health is ordinarily slight; often there is no effect. Yet intense odors may cause reflex actions of the digestive, muscular, and secretory systems. A vile odor may make one sick to his stomach or cause him to close his nostrils with his fingers, or run away. Even bad odors, however, do not cause disease, as the term is ordinarily understood; they do not shorten life. The irritants may cause smarting sensations and respiratory discomfort and probably are more closely related to health than true odors. Offensive odors may cause people to refrain from breathing deeply, may cause loss of sleep, restlessness, loss of appetite, and general malaise. These are certainly matters which appertain to health, and to some people they are of serious import.

If, however, the word, health, is extended to include the psychological as well as the physiological, if it includes human comfort as well as bodily functions, then odors do have an important influence on health. In addition to what has been mentioned, odors have strong powers of suggestion and give pleasure, discomfort, or disgust, as the case may be. They may interfere with one's quiet and repose.

Psychologically, people recognize the appropriateness of certain odors to times and places. The smell of fried onions in the kitchen may cause no complaint, but the same smell in the living room or sleeping room is objectionable, at least until nasal fatigue makes one used to it. The combination of nasal fatigue and a sense of the appropriateness of odors makes working under various conditions bearable, in a sewer, a slaughter-house, a garbage plant, a snuff factory, a candy store. Furthermore, such odors do not appear to affect permanently the health of the workers. The odors of a tannery in a residential district, of an oil refinery near a beach resort, of a garbage plant, sewage works, or rendering establishment in or around human habitations, and of a house drain in a living room, are regarded as inappropriate and are quite certain to cause complaint. Psychologically, also, it makes a difference whether the odors are local. In a village where all the people get their living from a pulp mill, no one seriously objects to the pulp-mill odor; but let a pulp

mill move into an established community, and the residents not connected with the establishment will regard the odors as a nuisance. Some odors also are regarded as a necessary evil, such as the smell of manure around the farm, and the smell of fertilizer on the garden; while the same odor suddenly appearing from an unknown source causes complaint. People have become accustomed to the smell of coal-burning furnaces; they have not yet become accustomed to the smell of oil-burners.

People object to certain odors because of their suggestiveness, and this raises the question as to whether bad odors may be taken as an index of unhealthful conditions. There seems to be only one answer to this, namely, that an odor is a useful, but not always a safe, guide. The sense of smell in animals is utilized largely in their search for food. In human beings, it is used in the selection of food. Human beings rebel at the odors associated with putrefaction, a matter of instinct which warns against eating unsafe food. Odors of decomposition in drinking water are also warnings which should be heeded. The odors of the algæ, however, are objected to not because they suggest that the water is unsafe, but because they are out of place. Where food and water are concerned, the warnings of bad odors should be heeded, but the absence of bad odors is not a proof of safety. Odors as guides to unhealthful atmospheric conditions are much less trustworthy. The extremely minute particles of odor-producing substances will travel through the air to far greater distances than bacteria, which, although tiny in themselves, are vastly heavier than the odor particles. It has been proved that the odor of a sewer or house-drain in the air of a room is practically no indication of the presence of bacteria from the sewer or the drain. It has been stated that such odors may be accompanied by minute quantities of chemical substances, which in larger doses are dangerous to the human system, but it has not been proved that these minute quantities are dangerous to health, although on this point Science has not yet spoken the final word.

The odor of coal-gas in a room has come to be regarded as an index of danger, the real danger being the odorless carbon monoxide; but water-gas, which contains much more carbon monoxide than coal-gas, has much less odor, and, for this reason, gas poisonings are more frequent now than before water-gas was largely substituted for coal-gas.

In spite of its shortcomings as an index of danger, in spite of its limited physiological bearings and its erratic psychological relations, the sense of smell is one which cannot be ignored, and human beings continue to insist that health and comfort be kept together and not separated artificially, as for the purpose of scientific discussion. The "health and comfort of the people" is an expression which not only has a great human urge back of it, but which has been found to stand the test of the Courts. In the speaker's opinion, the word, "health", used as a basis of the exercise of police power, should be given its broader and not its narrower meaning. In this connection, it is well to remember the following words taken from the first public utterance of the first State board of health in the United States, that of Massachusetts, in 1869:

"We believe that all citizens have an inherent right to the enjoyment of pure and uncontaminated air and water and soil; that this right should be

regarded as belonging to the whole community; and that no one should be allowed to trespass upon it by his carelessness or his avarice or even by his ignorance."

Meteorological Factors.—Inasmuch as odors and nasal irritations are caused by actual particles (it matters little whether they are regarded as gaseous, liquid, colloidal, or solid), transported by the air from their source to persons affected, a study of odor travel is very largely a study of meteorology, especially air movement.

First, moisture in the air intensifies odors. It provides a solvent for certain substances which otherwise might not be given off at the source, and it more readily conveys the odoriferous substances to the nasal mucus. A dry air tends to harden the surface of the mucous lining and protect the olfactory hairs. A warm air also intensifies odor, as a higher temperature hastens evaporation and volatilization as well as the sensitiveness of the nerve endings. A low barometer also hastens evaporation. Rain and snow act as natural air washers and purify the air. Ozone is said to have an oxidizing effect, but little is known of this. It may be that there are also electrical conditions which have an influence on the almost molecular particles which produce odor. The greatest factor, however, is air movement, a factor which in itself is dependent on temperature, barometric pressure, humidity, and other physical conditions.

Of the air movements, first consider those gentle movements, unmeasurable by the most delicate meter, due largely to temperature difference. They can be observed by movements of smoke and dust particles and by cooling effects on the skin. It is these almost insensible convection currents which spread the odor particles from their source for short distances in all directions. Very few odors are strong enough to be carried far from their source in this manner. Next, in order, are the gentle variable breezes which waft the odors to distances of a few hundred yards. They are usually intermittent, and not always constant as to direction, but they cover a wide angle. At a distance one gets the odor now and then. Finally, there are the winds, stronger and more constant, which carry the odors in a definite direction for long distances. The angle in this case is narrow. This pollution of the air by odors is not unlike the pollution of the water of a lake by sewage. The pollution may spread out slowly like a fan or, more accurately, perhaps, like a cone, or it may be carried by a strong current in a long narrow streak.

The speaker is not sure but what the author is right when he says that odors cannot be subjected to mathematical analysis. May it not be possible to divide odors into groups of different magnitude, define them in terms involving the velocity of the wind, the angle of the sector, or cone of pollution, and the distance from the source? The angle of the sector is a function of the veering of the wind, that is, the deflections of the wind from parallel lines, due to all kinds of minor local causes, which have a greater influence when the wind velocity is low than when it is high, as the higher winds are controlled by larger and more general causes. No classification should be attempted, however, until many careful observations have been made.

Topography, of course, is an important element in odor travel. Air travels through valleys, "draws" through breaks in hills, and is obstructed in its flow by forests and buildings. In any mathematical formula, these factors would enter as a coefficient which would vary with local conditions.

The elevation of the odor source has an important influence on the travel of the odor. Odors escaping from tall chimneys may be unnoticed on the ground near the base of the chimney but may be very noticeable a mile away. On certain days, when the sun is shining on the ground, the air in the lower strata becomes heated and tends to rise; consequently, the odors become lost in the upper air. At night, the conditions may be reversed; the odors lie lower. The water surface of a lake or harbor influences the vertical movements of air in ways that are well known. The importance of vertical currents is more appreciated since the aeroplane has come into use. The humidity conditions at night are also more favorable for odor travel. The effect of damp, muggy days on the intensity of the odor nuisance is too well known to need further mention. Muggy conditions not infrequently accompany a lower barometer, a factor which also tends to facilitate odor production.

Governmental Control of Noisome Odors.—According to Judge Jeremiah Smith, late of the Harvard Law School, a nuisance is whatever the Court decides is a nuisance. Attempts have been made to define a public nuisance, but in the last analysis the decision goes back to the opinion of "twelve good men and true" in a particular case. As far as the law is concerned, an odor nuisance, if action is taken by the Government under police power, must be proved to affect the "public health, safety, morals, or the like". The modern tendency is to extend the police power to cover public welfare and to let the phrase "and the like", cover public comfort, quiet, and repose. Certainly, it will be admitted that bad odors and irritating fumes are a public discomfort. In such matters as these the opinion of the layman is often sounder than that of the expert.

American cities are growing, and as they become larger it is right that the police power should be extended and applied more severely. A slight odor affecting a large population may be as much of a nuisance as a very bad odor affecting a small population; for in the large population there is a greater chance that more persons susceptible to the slight odor may be present.

Bad odors which are spread over large residential areas, may depreciate property values, but the speaker fails to understand how this can be properly regarded as coming within the scope of the police power. In board of health hearings, however, this argument is commonly introduced. Claims for damages to property are for the Civil Courts to adjust. Lawyers know, however, that police power action by a health board is a help in a suit for damages, and not infrequently this is the motive back of nuisance complaints. One of the greatest difficulties in the governmental control of nuisances is to know whether the complaints are really sound.

Another serious difficulty in such control of objectionable odors from industrial works is that the nature and extent of the odor and the certainty of its being a nuisance cannot legally be determined in advance. In the Staten Island garbage case, referred to by Mr. Tribus, the Governor of the

State of New York decided that the State's power of abatement could not be exercised until the works had been completed and had been found actually to be a nuisance, in spite of the fact that an investigation by the State Health Commissioner had shown that a nuisance would probably occur. This brings up the necessity of the exercise of greater care by local authorities in granting building permits for industrial plants. Local governments are usually glad to have a new industry established and readily grant a permit for building, often without bringing the matter to the attention of either the local or the State board of health. In the case of chemical manufacturing, the permitted establishment may create an odor which causes widespread and justified complaint. Then comes the difficult decision. Shall an established industry be closed? Shall the economic advantage to the community be tacitly considered in the exercise of police power? Shall economic advantage be considered as offsetting public discomfort? Or, if the case is one in which the odors can be prevented, how far shall the manufacturer be made to correct them? And if the industry is a new one, how long shall suffering citizens be made to wait while new devices for odor reductions are being tried? The lesson taught by many unpleasant experiences of this kind appears to be that it is wise to exercise greater caution in the location of industrial works which involve chemical processes, and to limit them to certain districts or zones.

The speaker was a member of the New York Commission on Building Districts and Restrictions, and his studies at that time led him to believe very strongly in the zoning idea. Zoning alone, however, is not enough. The Staten Island garbage plant was located in an established industrial district, but the trouble was that the odors "would not stay zoned". One way of avoiding the difficulty would be to require local authorities, before granting a building permit to a chemical industry, or any industry likely to cause public offense, to receive a letter of advice from the State department of health or other competent authority, such letter to be published and thus brought to the attention of the people before action is taken. Although the layman is as good a judge of what offends his nose as a sanitary expert, the expert is better able to judge in advance of the probability of a public nuisance being created. No such advisory action is needed in regard to the location of certain nuisances which have been adjudged by the legislature or the Courts to be nuisances *in esse*. Certain industries have well earned the title of "offensive trades", abattoirs, soap factories, rendering works, tanneries, and the like, as well as sewage works and garbage works. Already, the laws in some States require the location of such works to be approved by a sanitary authority. Just as health departments establish lists of diseases dangerous to the public health, so should they establish lists of industries which threaten the public health and comfort.

When a manufacturing plant, by disseminating irritating fumes or odors, becomes a public nuisance, the demand is usually made that it be shut down, heavy fines levied, or some other penalty inflicted. There are cases where such punishment is just and where the nuisance cannot be abated in any other way. In many instances, however, the elimination of odors is a difficult matter, and those who operate the plant do not know what to do; they do not feel

sure that the expenditure of money for certain devices will accomplish the result. In other words, what they need is sound advice, not punishment. Unfortunately, the Engineering Profession has never developed specialists in odor prevention. This appears to be a field worthy of serious consideration. Combined with smoke and dust prevention and ventilation, it should furnish ample scope for a career for a young man who has a knowledge of chemical engineering, physics, meteorology, and sanitation. For many years State departments of health have been in a position to give advice in regard to the purification of water and the disposal of sewage and, in the speaker's judgment, the use of this advisory power has accomplished more than the sterner measures of the police power. The disposal of sewage, however, is practically a single problem, although, of course, it has its complications, and it is also a public problem. In the case of chemical manufacturing, the situation is different. The problems are very complex, the works are privately owned, and the processes are often secret. Hence, the consulting specialist is in a better position to handle the problem than the sanitary department of a State board of health.

Not infrequently, odors from industrial plants are due merely to bad management, carelessness, or sloppiness, and the correction of these faults often leads to greater profits to the concern. The escaping fumes may be waste products, but they are sometimes valuable and worth saving.

New lines of manufacturing are constantly being taken up and new problems are continually arising. Thus, in Massachusetts, several oil-refining plants which represent investments running into several million dollars, have recently been built. Some of these plants are handling Mexican oil which contains much more sulphur than the common American products. The refining processes have given rise to odors and irritating fumes of an offensive character, which, in one notable instance, spread over a wide area and caused a serious public nuisance. Recognizing the novelty of the problems to be solved and the importance of the industry to the community, an order requiring the works to close was stayed from time to time, in order that the effect of new devices might be ascertained. Finally, however, there appeared to be no other course, just to the people, than that of closing the works until certain advised alterations could be made.

Odor Prevention.—This leads naturally to the subjects of odor prevention, air purification, and air analysis, which are too detailed for the speaker to discuss. Various methods are now available. The sedimentation of dust, the electrical precipitation of dust, acid, fumes, etc., by the Cottrell system, the washing of air by drops and sprays of water, the burning of organic dust at high temperatures, the filtration of air through cloth, the ventilation of buildings, the removal of odors absorbed by floors, walls, and fabrics, by washing and by the use of a compressed air blast, the use of disinfectants to prevent or arrest decomposition of organic matter, the application of sunlight, the use of chemicals to alter the character of the odor-producing substances—these and other methods, used singly or in combination, as the situation may demand, furnish means by which almost any industry may be made amenable to reasonable standards of health and decency.

STEPHEN DEM. GAGE,* ESQ.—Both the author and Professor Whipple have pointed out that much trouble might be avoided if some method were devised for locating odor-producing industries where they would be least offensive. The situation which exists to-day in certain parts of Providence, R. I., and the adjoining communities, is the direct result of such lack of foresight.

About ten years ago the Chamber of Commerce and other public bodies endeavored to increase the importance of the Port of Providence, and a number of the large oil companies were induced to establish extensive receiving and distributing stations at the head of Narragansett Bay. To-day, Providence is one of the largest oil ports in New England. In connection with the operation of these oil stations, two companies established plants for the separation of crude oil into its component parts by distillation, and at one of them practically all the processes used in the refining of oil products are carried out. These two plants are about 3 miles apart, on opposite sides of the Providence River, and within about 4.5 miles from the center of the city.

Unless plants of this kind are completely equipped with special devices to control odors and are carefully operated, offensive nuisances are likely to arise from odors which at times may travel considerable distances. Shortly after these plants began operation, complaints of offensive odors in adjacent residential districts were received, which increased both in number and in vehemence, until the Legislature at its last session directed the State Board of Health to investigate and report on the subject. During the progress of this investigation much has been learned about odors of this kind and about their travel habits. Since the results were to be reported to the Attorney General for action, there are many interesting points which cannot be discussed at this time.

The odors from oil-distilling and refining plants are probably of a mixed character, partly gaseous and partly colloidal. They are variously described as "burnt oil", "burnt rubber", or "sweetish burnt rubber." Observers who have had laboratory experience and chemists generally use the latter term as most definitely expressing their impressions. The character of these odors varies considerably with the dilution. Near the source when they are intense, they have the characteristic odor of rotten eggs, or sulphide odors. As the distance from the origin increases, and they become more dilute, they take on the odor of burning rubber, with a sweetish or slightly burnt sugar flavor. With more dilution the rubber constituent disappears, and the odor may best be described as that noted when a baker's oven full of well browned bread is opened. When still more dilute, these odors are like that of slightly scorching paper. At first, these changes in the character of odors from the same source led to considerable confusion. In one instance the speaker located a not unpleasant odor on the leeside of a large bakery, which was attributed to that source, and was surprised to find that this odor was much stronger on the windward side of the supposed source and it became decidedly offensive as it was traced down to its origin.

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The reason for this change of character in odors is undetermined. It may be that odors are a blend and that one component is diluted out sooner than others. It is well known, however, that some apparently simple odors change in character with dilution. The characteristic odor of kerosene is recognized easily by almost every one, but when greatly diluted it is more likely to be described as some kind of perfume.

It may be of interest to mention that considerable care has been taken in order to obtain comparable observations from different observers. The acuteness of the olfactory sense varies widely in different people and in selecting observers for this work, men were chosen whose sense of smell was reasonably alike. These men were then taught to record odors of similar intensity in a similar manner. The scale was the same as for recording odors in water analysis, "very faint", "faint", "distinct", "strong", and "very strong", the distinct odor being one which would be readily detected by almost every one and which would be disagreeable to most people.

From observations the speaker believes that the travel distance of odors of this character is mainly a function of two factors: the amount of odor produced at the source, and the velocity of the wind. There are other factors, however, which apparently enter into the problem. Weather conditions affect the travel habits of odors to a certain extent. The theory that sunlight may have a deodorizing effect may account for the fact that odors of this kind have been more troublesome at night or on days when there was a fog or heavy mist, than on clear bright days. As instances of odor travel, the following observations might be recorded:

1.—Night, weather clear, wind from 3 to 4 miles per hour, travel distance about 1.5 miles, area affected about 3 sq. miles.

2.—Day, rain, wind about 4 miles per hour, travel distance about 4 miles. In this instance, the odor was carried across the water about 2 miles before affecting a residential area of about 2 sq. miles.

3.—Night, cloudy with light showers, wind from 4 to 5 miles per hour, travel distance about 2.25 miles, area affected about 5.25 sq. miles.

4.—Night, cloudy, wind about 5 miles per hour, travel distance about 4.5 miles, area affected nearly 10 sq. miles.

5.—Night, weather clear, wind about 9 miles per hour, travel distance about 1.5 miles, area affected approximately 3 sq. miles.

6.—Night, weather clear, wind about 9 miles per hour, travel distance about 2.25 miles, area affected about 5 sq. miles.

One other interesting observation may be mentioned in which the direction of the wind was such that strong odors from two sources mingled and fortified each other. The characteristic odor was noted at a travel distance of 2.5 miles from the nearest source and about 6 miles from the more distant source. At this time a residential area of about 11 sq. miles was affected, in addition to an undetermined area over the water. This odor occurred at night in clear weather and with a wind velocity of about 5 miles per hour.

It should be noted that the wind velocity is that recorded at the Government Meteorological Station, 2 to 6 miles from the areas in which the observations were made. These figures are the only ones available, but the

possibility that the wind velocity might be different in the two localities must be taken into consideration in any interpretation of the observations noted.

The author has pointed out that there are many apparent peculiarities about the travel of odors. With little or no wind, they do not appear to travel far from their source, and if the wind is strong, they are apparently dissipated and not troublesome at a great distance. In some instances they may travel high, although at other times, with apparently similar conditions, they will travel low. There are a number of observations where these odors, quite offensive a mile or more away, could not be noted near the point where they originated. In one instance an odor was traced from hilltop to hilltop for a considerable distance, but could not be detected in the valleys. Other cases have been observed of an odor settling in air pockets and remaining an offence long after it was discharged from its source. This is most likely to occur on damp, foggy nights with little wind. It has been also frequently observed that these odors will be strong on certain streets and perhaps cannot be detected in parallel streets on either side. Variations in odor intensity in near-by localities can be attributed only to differences in air currents. Such variation may be responsible for a wide difference of opinion among the residents of certain sections as to whether a certain industry is offensive or non-offensive, and such differences must often be proved and reconciled before relief from offensive conditions can be obtained.

Another phase of the oil problem becoming prominent, which may become of greater and more far-reaching importance than odors from oil-distilling and refining plants, is the probable effect of the substitution of oil for coal in power and heating plants. There is no question that oil fuel is easier to handle, easier to store, and more economical than coal. The bulk of fuel oil used in New England cities at present is from Mexican crudes which carry about 4% of sulphur. The fuel oil contains more than 4% of sulphur since much is left behind when the naphtha and gasoline are removed. If the combustion of this fuel is complete, the sulphur is converted into sulphur dioxide, an irritating gas which is discharged into the air. If the combustion is not complete, which is frequently the case, a part of the sulphur passes off as hydrogen sulphide or other sulphide gases, together with some partly burned oil, all of which are extremely offensive. Many oil-burners have been constructed in fire-boxes designed for soft coal and it is practically impossible to obtain complete combustion. Furthermore, some of the oil-burners now on the market are defective and cannot be operated to the best advantage.

For purposes of illustration, assume a community using 1 000 000 bbl. of oil fuel per year. This is not unreasonable and the speaker knows of a single power plant now using this quantity. Only 4% of sulphur would amount to about 30 000 lb. of sulphur burned per day. With complete combustion this would be equivalent to 60 000 lb. of sulphur dioxide gas discharged from the stacks every day. The speaker does not know that this sulphur dioxide when diluted with the air would directly affect the health of the people; he

does believe, however, that if the oil-burning plants were concentrated and near enough to the residential section, it would cause more or less serious irritation to the nose, throat and lungs of a certain portion of the population, and perhaps render these people more susceptible to other diseases. The speaker also believes that under prevailing conditions at many oil-burning plants to-day, the odors of incompletely burned oil and sulphides would be so frequent that this community would not be counted a desirable place in which to live. This is the problem to be solved in the near future. This problem and all odor problems can be solved, but the engineer and the chemist must first build a solid foundation of scientific facts before anything definite can be accomplished.

I. S. OSBORN,* M. AM. SOC. C. E.—The subject presented by the author is a timely one, to which practically no attention, study, or research has been given. Had a small part of the cost of litigation in connection with suits brought to eliminate nuisance from odors been expended in study and research, there would have been accomplished far more toward the elimination of odors than has been achieved by Court decisions.

Since the speaker has had more or less intimate contact with all the plants mentioned by the author, the points raised in his paper relative to garbage disposal works and the odors produced are of especial interest.

In dealing with the problem of garbage reduction, one must necessarily consider somewhat the size to which this industry has grown in the United States. In an investigation for the United States Food Administration in 1918, the speaker found that 29 of the larger cities disposed of their garbage by the reduction method, and produced annually 72 000 000 lb. of grease and 150 000 tons of fertilizer tankage, valued at $11 000 000. These figures demonstrate that the industry deserves considerable attention. The nature of the business, as well as the attitude of the men or the companies engaged in the operation of disposal plants, has not given much incentive to develop the industry along scientific lines, with the result that it has grown to this size without much assistance from the Engineering or Chemical Professions.

The plants have not been perfect either for the making of all possible recoveries or the elimination of nuisance from odors. However, the industry should not be condemned, but should have the efforts of engineers and chemists toward development whereby it can be operated without nuisance and without the objections due to past performance.

In this discussion of garbage reduction plants the odors considered are those which will travel or give rise to complaint in surrounding communities.

The reduction methods for garbage disposal vary in detail, but the majority of plants use the digester and dryer systems, in which the garbage is cooked in digesters, with live steam, after which it is pressed and dried.

In plants of this type the sources of odors are from the raw garbage, exposure of material during process, finished products, leaks in apparatus, vents from digesters, and gases from direct heat dryers. The greatest source of objectionable odors are the gases vented from digesters, or the gases

* Cleveland, Ohio.

from the dryers. The vent gases are the most penetrating and objectionable, but are more easily controlled and treated than the much larger volume of sweetish or caramel odors from the dryers. The odors from the raw garbage, the plant, and the finished products can be controlled and, with proper design and operation, should not give cause for nuisance.

In 1915, the speaker made an investigation and report to the Committee on Street Cleaning, Board of Estimate and Apportionment, New York City, on the elimination of odors from the Barren Island Disposal Plants. At the time of this investigation practically no data on this particular subject were available. This investigation covered the three plants operating at that time on Barren Island, namely, the plants of The Sanitary Utilization Company, which were then used for the disposal of garbage under contract with New York City; the plant of the Thomas F. White Company, which was operated for the production of tankage by drying the digested garbage; and the plant of The Products Manufacturing Company, which, under contract with the City of New York, disposed of dead animals and meat offal, and also rendered garbage collected from hotels and restaurants.

The assistance of The Central Testing Laboratory, maintained by the city, was obtained for analytical work in making these studies. The investigation which covered a period of about four months was for determining the necessary steps to remedy and eliminate the objectionable odors from these plants. Tests were made to determine the source of the odors, the volume of the gases carrying the odors, as well as the methods that might be adopted to deodorize the gases. The tests were made both in the field and in the laboratory, and, except the chemical analysis of gases which was made in the laboratory, the larger part of the work was done at the plants while they were in operation.

These tests involved the development of methods in regard to which no precedents and data were available. It was found that the apparatus secured for this work was not adaptable and it necessarily had to be remodeled, calibrated, and tested. Search was made for information on this subject as well as on kindred subjects. Although the work was not carried to the extent desired, the results of this investigation were, in many ways, satisfactory, many facts were established, and information of great value was gained regarding the design and operation of such plants.

Analyses of the gases given off from the vents of digesters before and after passing a barometric condenser showed that the carbon dioxide, which amounted to 45.2% before passing the condenser, was reduced to 6.2% after passing it, which indicates the efficiency of the condenser in absorbing condensible gases. The odors from digesters were found to be due primarily to the sulphur compounds and to a less extent to essential oils.

Tests to deodorize the vent gases entirely eliminated the odors by washing the gases in a solution containing small quantities of sodium and calcium hypochlorite (chloride of lime). It was found that 10 cu. ft. of gas could be deodorized in 150 cu. cm. of calcium hypochlorite solution containing 4 parts of available chlorine in 1 000. Gases treated by heating alone were not deodorized, but when heated to 1 100° Fahr., the sulphur compounds changed to sulphur dioxide which was soluble in water; so that by washing after

heating, complete deodorization was obtained. The commercial application of the calcium hypochlorite solution in washing gases was difficult in practical operation, and the method of heating and then washing showed the greatest possibility of application.

Since this investigation, developments have been undertaken whereby gases carrying odors are treated with chlorine gas. The results have not yet been determined by analysis, although where the chlorine gas has been applied to digester vent gases, the characteristic odor is not noticeable at the point of discharge. It has not been determined whether the odor is disguised or masked by the chlorine gas or whether chemical action deodorizes the gases.

Where the direct heat dryers were used, the volume of gas given off was excessive. From 175 000 to 200 000 cu. ft. of gas per min. escaped from the dryers in the plant of the Sanitary Utilization Company. The cause of odors in dryer gases is due largely to the scorching or burning of the material and a sweetish and caramel odor is produced, due to the scorching of sugars. In making tests, dryer gases were completely deodorized by washing, the elimination of odors by scrubbing being in proportion to the degree they were washed. It is difficult in washing gases to eliminate the odor entirely, although it is possible. Heating the dryer gases to a temperature of 1 800° Fahr. eliminated the odor; they were also completely deodorized in calcium hypochlorite solution.

Experiments to ascertain whether the odor could be eliminated by the dilution of gases with air, showed that the odor was still distinct in 1 part of dryer gas with 400 parts of air; that it is characteristic of this gas to travel more or less in pockets and not diffuse readily with the air; and that it is not feasible to depend on the dilution of dryer gases to eliminate odors.

The elimination of odors from handling materials, as well as plant and room odors, is a problem to be considered in the design of plants. With suitable methods adopted for their control, and attention in operation, they should not give rise to objection. It is fundamental, however, in the operation of disposal plants that the first object should be the elimination of the production of odor, thereby making it unnecessary to provide means for deodorizing.

The statement made by Mr. Tribus that, although it was possible to operate the Staten Island plant without offense, it was not practical, is not borne out by fact. The financial results obtained by the company would have been greater if the plant had been operated without offense.

The Staten Island plant uses the Cobwell process, which differs from the digester system. In the digester system the garbage is sealed in tanks or digesters and live steam is admitted for cooking. The digesters must necessarily be vented to insure a supply of steam for raising the temperature of the material. After digestion, the material is pressed, and the solids are dried. In the Cobwell process a reducer with a steam jacket is used and the garbage in this reducer is submerged in a solvent for the grease, which acts also as a dehydrating agent. Water or steam vapors combined with solvent vapors leaving the sealed reducer pass through a vent pipe to the condenser,

where they are practically all condensed. Tests showed that the vapors given off from the reducers, after passing the condenser, were odorless.

There is no question regarding the odors referred to by Mr. Tribus from the Staten Island plant. From the speaker's observation they were detected at a distance of from 3 to 4 miles. The odor was not the same as that from the plants on Barren Island, where the digester and dryer systems were used. In all probability, most of the odor given off was the result of the operation of the plant without proper sealing, which allowed the gases to escape directly to the atmosphere without passing through the vent line to the condenser. This is borne out by the results of investigation as well as from operation, and is the cause of the great loss in solvent which resulted from this practice.

The speaker was in charge of this plant during the first two months of operation, during which time it was found, not only possible but practical, to operate without producing gases having odors. During this time, from daily inspection and study, the odors emanating from this plant were never noticed off the property on which it was situated. The experience of the company in regard to loss of solvent demonstrated that it did not pay to permit odors to escape, for with the escape of any gas from the reducer there was a corresponding loss of solvent.

The statement made by Mr. Tribus that slaughter-houses have gradually and almost ceased to be offensive is not fully borne out by investigations of this industry. In many cities, as in former years, they are either tolerated as a necessary evil, or there is continual agitation and occasional litigation in regard to them.

Mr. Tribus states that "the real point of importance, however, is the distance that smells travel and their actual effect on human beings". Is it not of greater importance to determine the source and the necessity of the odor. In most cases, the source can be determined and the emission is unnecessary.

A survey of the past efforts and policies of municipalities with reference to this subject, reveals a lack of attention given to the problem. As long as municipalities expend no greater effort toward an engineering solution of the garbage disposal problem in the design of municipal plants, and continue the practice of awarding short-term contracts for the disposal of garbage, whereby the contractor must charge off his investment during this short term, attempt to pay dividends from the earnings under the contract, and, at the same time, meet competition as to price, just so long will the same results be obtained as at present, from a large number of plants.

This statement should not be taken to mean that all disposal works should be municipally operated, but each municipality should control not only the operation but the initial outlay in order not only to safeguard the city against objectionable methods, but from an economic standpoint as well. The continued unsanitary results, agitation, and litigation that have accrued are only natural, and so long as present practice is continued, so long will objections and litigations continue.

The statement by Mr. Tribus that the garbage reduction plant on Barren Island was "for years synonymous with stench", is true, but he is mistaken in that the plant was closed due to legal action. It was closed because the owners

did not secure a renewal of their contract with the city. An injunction, however, was granted forbidding the operation of The Products Manufacturing Company's plant on Barren Island, where, under contract with the City of New York, dead animals were rendered. The injunction prevents only operation whereby a nuisance is created, and the plant is still operated, and from all intents will continue to be operated until it is proven a nuisance. Since this injunction was granted the City of New York has awarded a new contract to the same company for the disposal of animals.

It would seem that greater results could be obtained if the public would look to the city for improvements and relief rather than to the Courts. "Control by anticipation" raises a difficult question and it is doubtful whether little would be gained, as compared to the results that could be obtained if the responsibility for securing the best method were placed on the municipality.

The developments to date in the elimination or non-production of odors have not resulted from any powers that could be granted pertaining to control by anticipation, but from adverse rulings and Court decisions, making it necessary to improve conditions in order to safeguard investments. There is a possibility that control by anticipation would have a tendency to retard development.

Little was known about conducting garbage reduction plants until the municipalities entered the field. Some advance has been made, from necessity more than from actual study and attention to the development of the systems. When the same attention is given to this problem, as to many other municipal problems, and plants are constructed as permanent investments, advancement may be expected.

RUDOLPH HERING,* M. AM. SOC. C. E. (by letter).†—The writer is pleased that the author has presented the results of his experience in regard to a subject which has been surrounded by much indefiniteness, ignorance, and prejudice. More facts based on reliable data were needed to form a fixed foundation for judgment, some of which the author has supplied.

Sentimental and personal perception play a large rôle in this subject; but thorough analyses and more impersonal data will help to reach useful conclusions.

Seven years ago, the writer presented a paper before the American Public Health Association on "The Prevention of Odors at City Refuse Disposal Works". As some of the data contained therein may contribute toward an explanation of this subject, a few of them will be repeated.

An odor is perceived by the stimulation of a small area, less than that of a one cent piece, near the upper end of the nasal duct. The normal excitation is caused by the impingement thereon of molecules of matter, either in a solid, colloidal, liquid, or gaseous form. That odorous air absorbs more heat by the addition of these molecules than pure air, was proved by Tyndall. That its specific gravity was increased sufficiently to retard the diffusion was

* Upper Montclair, N. J.

† Received by the Secretary, November 16th, 1921.

proved by Tigerstedt, who likewise proved that odorous vapors are condensed by absorption at the surface of glass, paper, water, skin, and clothing. Odor molecules detach themselves from the surfaces by evaporation, oxidation, or by hydrolytic decomposition, and may be carried some distance in currents of air.

Some scientists have believed that only gases stimulate the olfactories. Others have demonstrated that liquids, such as sulphate of magnesium, also convey odors. Sharks have a well developed sense of smell.

Odors can be perceived more readily when the air is moist than when it is dry. A dry membrane in the nostrils cannot detect odors, nor does it always respond when the nostrils are completely filled with a liquid, even with eau de cologne. Some persons are incapable of detecting certain odors, such as vanilla, violets, and some faintly burning vegetable matters. The olfactories can tire of some odors and fail to respond; and yet at the same time they can perceive a sudden appearance of other odors. Nagel found that two or more entirely different odors may cause a composite odor, entirely different from either of the originals. Four grammes of iodoform can be made almost inodorous by 200 mg. of Peru balsam, as proved by introducing the two substances separately into the two nostrils. It was also found that the odors of paraffin introduced into one nostril will completely neutralize the odor of rubber, when a certain portion is introduced into the other nostril.

Prejudice has terminated in lawsuits, at times, as evidenced by complaints made of odors of burning garbage, when the actual cause was neighboring plants producing other offensive odors and when, in a new plant, smoke was seen to rise from the incinerator chimney and the plant was only being tested with preliminary coal fires.

Further, it must be realized that the odor particles are of different sizes. They are largest when in solid and smallest when in gaseous form. The odors escaping from incinerators are due usually to unburnt solid particles. As long as they drift in the air current from the chimney they will be noticeable, even for miles, in the form of, as it were, invisible clouds. The writer remembers such a case in Staten Island when suddenly in a clear sky and on an elevated bluff a strong burnt-garbage odor was apparent, brought by a gentle wind current straight from a refuse incinerator about two miles distant. It is evident that in discussing the question of odors many fundamental facts must be considered.

The intensity of odors is affected by temperature. If organic matter is burned at about 1 800° Fahr., no odor will result, nor is such matter offensive when it freezes. The most intense odor generally results between temperatures of 90 to 150° Fahr. Dryness likewise controls odor intensity as has already been stated. In a dry climate, odors disappear rapidly.

Odors arising from gas particles which can easily decompose in the air, such as sulphureted hydrogen, disappear more quickly than when they arise from solid or colloidal particles.

From splashes or sprays the odors from sewage are increased when not only gaseous, but also colloidal, particles are thrown in the air, and drift quite a distance. Air from filters, sprinkled with putrefying sewage is often foul-

smelling and is noticeable much farther than the air drifting over a quiescent sewage bed. Dilution of the foul air from the dryers at garbage reduction works, in the proportion of 1 to 500, has destroyed its odor.

When sewage containing odorous gases in solution or suspension, is discharged in streams, an odor has been perceptible near the water surface, even several miles below the plant. The odors after their escape, however, disappear very quickly, as they consist chiefly of gases, and not of colloids or solids.

As already stated, odorous particles suspended in the air, when adhering by absorption to surfaces of walls, clothing, etc., may persist for a long time. They can be removed by brushing and by washing down with blasts of compressed air directed against the surfaces. They can also be removed from clothing by exposure to sunlight.

From the facts known at present it is possible to reduce materially, and even prevent entirely, the odors arising from refuse disposal and sewage works, by a careful study of the specific conditions.

OLIN H. LANDRETH,* M. AM. SOC. C. E.—A consideration of the travel habits of odors will readily indicate that they are lawless creatures, going with the wind which "bloweth where it listeth"; still, in their travels, they must be placed in the class of "short haul" rather than "long haul" traffic.

However, in this "short haul" travel, odors are no respecters of persons, nor of boundary lines, either private, municipal, or State, and, as a consequence, are occasionally found in jurisdictions to which they are not native, and even sometimes in adjoining States.

Since odors may be disagreeable, or offensive, or sometimes even seriously injurious to persons or property interests, they are frequently proper subjects for repression and abatement. When the occurrence of odors becomes serious enough to be a violation of law, whether common or statutory, that law may be invoked to repress those odors. Such legal repression or abatement is not always a simple or easy matter, even when the injury is suffered in the same legal jurisdiction as that in which the odors originate, but when the cause and the effect lie in different legal jurisdictions, and especially in different States, the problem of abatement is a still more serious one. It is not easy to reach across a State line and repress an odor.

Two instances will be cited where odors in their travels have ventured across State lines and have been abated by legal measures; in the first, by the legal remedies constitutionally provided and usually followed in such cases; in the second by creating, under what were rather unusual conditions, a novel form of legal relief, more expeditious and apparently as satisfactory in results as the usual form.

The first case is that of the fumes from the plants of two copper smelting companies at Ducktown, Polk County, Tenn., near the southern border of the State, which fumes were the cause of serious injury to property interests across the line in the State of Georgia.

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* New York City.

The efforts of the injured parties to obtain relief by private litigation proved unsuccessful, and the State of Georgia finally sponsored the cause of its citizens who had suffered injury and, in October, 1905, brought an action for injunction against the two corporations in Tennessee responsible for the production of the injurious fumes, which, having destroyed all vegetation in sight in Tennessee, crossed over into Georgia and caused the injury to vegetation and animal life there complained of.

The defendant corporations were the Tennessee Copper Company and the Ducktown Sulphur, Copper and Iron Company. As one of the parties to the litigation was a sovereign State, the case was necessarily brought as an original action in equity before the United States Supreme Court.

The Tennessee Copper Company's plant is about ½ mile from the Tennessee-Georgia line, and the Ducktown Sulphur, Copper and Iron Company's plant is about 2½ miles from that line. The injuries suffered were from large volumes of sulphur dioxide discharged by both plants, originally from the ground-roasting of the ore, and, later, from the smelting furnace chimneys.

The copper ore used by both companies was a mixture of pyrrhotite and chalcopyrite carrying slightly less than 2% of copper and about 20% of sulphur. The Tennessee Company used about 450 000 tons of this ore per annum, and the Ducktown Company about 200 000 tons, and prior to the filing of the action, the two defendants probably discharged into the atmosphere about 240 tons of sulphur per day.

Like all original actions before the U. S. Supreme Court, testimony was taken before a commissioner, the hearings were widely separated and long drawn out, and at irregular intervals the Court itself heard motions, made rulings, delivered opinions, and finally in the case of the Ducktown Company, issued a decree granting a partial injunction which required the company to restrict the discharge of gases to the equivalent of 20 tons of sulphur per day during the growing season from April 10th to October 1st of each year, and to 40 tons per day during the remainder of the year.

In the case of the Tennessee Copper Company, the Court recognized a stipulation entered into with the State of Georgia, by which the State of Georgia agreed to refrain from asking a decree of injunction prior to October, 1916, on condition that the Copper Company would: (*a*) provide a fund annually to compensate injured parties in Georgia; (*b*) conduct its plant subject to inspection in specified ways; and (*c*) between April 10th and October 1st of each year limit the smelting of green ore to such amounts that the sulphur dioxide therefrom could be taken care of by its sulphuric acid plant when working at its normal full capacity. This arrangement between the State of Georgia and the Tennessee Copper Company still continues, and the case before the Court is continued indefinitely, no decree having ever been issued against this defendant. The awards of damages to Georgia claimants, the administration of the indemnity fund, and the compliance generally with the terms of the stipulation, are determined by a permanent commission of three members, of which one is chosen by the Tennessee Copper Company, and one member and an arbitrator or umpire by the State of Georgia.

During the past three years the defendant Ducktown Company has also been permitted by the Court to operate under this same stipulation conjointly with the Tennessee Copper Company. Each of the two defendants contributes annually to the indemnity fund a fixed proportion of the total awards made.

Thus, after many years spent in litigation before the U. S. Supreme Court, without including the previous period of litigation as private individuals, a reasonably satisfactory adjustment was finally reached.

The second instance cited of inter-state odors is that which Mr. Tribus simply refers to, namely, the case of the West End Association and the City of New York, Plaintiffs, *versus* The Barrett Company of West Virginia, The Corn Products Company, The General Chemical Company, The Midland Linseed Products Company, The Valvoline Oil Company, The Bulls Ferry Chemical Company, and Spencer Kellogg and Sons, Defendants.

This case is cited solely because of the very novel and original form of legal remedy invoked. During and before 1916, the defendant corporations which had manufacturing plants at or near Edgewater, N. J., on the west bank of the Hudson River opposite West 80th Street to West 110th Street, New York City, were, or at least some of them were, producing at their New Jersey plants offensive odors. These odors were blown across the Hudson River and greatly annoyed the citizens of New York City living on or near the east bank of the river opposite the plants in question. An action had previously been brought in 1915 by the Attorney General, representing the State of New York, before the U. S. Supreme Court for an injunction to restrain the offending New Jersey corporations from discharging the objectionable odors. Beyond having some investigations made by the State of New York, the case was never prosecuted and still lies dormant. A strong local civic organization called the West End Association being already active in the locality affected by the fumes, took up the matter for the citizens. Instead of resorting to the usual litigation, either as private plaintiffs before the Federal District Court, or through the case already brought by the State of New York as plaintiff before the U. S. Supreme Court, as was done in the Georgia-Tennessee copper case cited, and in the New York *versus* New Jersey Bayonne fumes case, also referred to by Mr. Tribus, the West End Association developed the following form of relief.

An Act of the New York Legislature was procured, being Chapter 292 of the Laws of 1917, amending the general corporation law which constitutes Chapter 23 of the New York Consolidated Laws. This amendment, which became Article 9-A of the General Corporation Law, provides in Section 200:

"That any domestic or foreign corporation which shall so conduct its business without the State by the emission or discharge of dust, smoke, gas, steam, or offensive, noisome or noxious odors or fumes, so as to unreasonably injure or endanger the health or safety in this State [New York] of any considerable number of the people of this State, shall be deemed guilty of a nuisance, and the charter of such corporation if incorporated, or formed by or under any law of this State shall be deemed forfeited in the manner prescribed in this Section, or its certificate of authority to do business in this State [New York] if incorporated or formed under the laws of any other State shall be declared revoked and annulled in the manner prescribed in this Section and in either case shall not be revived except as prescribed in the next Section."

The law also provides the following procedure: Complaints shall be made to the State Commissioner of Health. He shall cause a copy thereof to be served on the corporation complained of, requiring that the matters complained of be abated, or that the charges be answered in writing within a time specified. If the charges are not thus satisfied, and if there appear to be reasonable grounds therefor by the State Commissioner of Health, he shall cause such charges to be investigated and shall fix a time for a hearing on such complaints. If the State Commissioner of Health after hearing and investigation shall find that such corporation is conducting its business without the State so as to injure or endanger unreasonably the health or safety in this State of any considerable number of people of this State, he shall file his findings in duplicate with the Secretary of State and with the Attorney General. A certificate of the Secretary of State giving notice of the filing of such findings shall be served on the corporation if domestic, or on the designated agent of a foreign corporation authorized to do business in the State, and thereupon the charter of such domestic corporation, or the right of a foreign corporation to do business in the State shall be suspended for thirty days. If at the expiration of that period, the State Commissioner of Health shall on further investigation and hearing render a finding that the nuisance is continued, he shall cause a notice of such determination to be served on the corporation or the designated agent of a foreign corporation, and published once a week for two successive weeks in the official State paper. On the tenth day of such service and publication, the charter of the said corporation if domestic, or the certificate of authority to do business in the State if a foreign corporation, shall be deemed to be forfeited.

Any person who shall attempt to exercise any powers under the charter or the certificate to do business, which has been so revoked or forfeited, shall be guilty of a misdemeanor. If the charter of a domestic corporation has been forfeited, the Attorney General shall apply to the Supreme Court for the appointment of a receiver of the property, who shall have all the powers and duties, as far as is practicable, which are prescribed by Articles 10-A and 11 of the General Corporation Law. The Act also provides for the reinstatement of a charter or of the authority to do business, after the nuisance has been discontinued, and after a suitable guaranty has been furnished that the corporation will not longer maintain such nuisance.

Equipped with this new form of legal relief, the two plaintiffs in this action, representing and acting for a large number of private individuals and, perhaps, some firms and corporations, who were suffering from the odors, commenced proceedings in the summer of 1917 against the defendant corporations under the new law before the New York State Commissioner of Health. Two separate investigations were instituted: The first by the plaintiffs to establish in some detail the facts on which to elaborate the charges, to establish the general feasibility of abatement, and to base the examination and cross-examination of witnesses; the second, later, by the State Commissioner of Health to supplement the testimony developed at the hearings, on which to predicate the findings. Under the first investigation, the inspection of the plants of the defendant corporations clearly showed that the occurrences

of odors causing offense in New York City could be grouped into two distinct classes: (1) those resulting from the usual, normal operation of the several plants, which, while fairly constant in character and volume at the plants themselves, nevertheless produced widely varying degrees of offensiveness in New York City, depending on the direction and force of the wind, and on the humidity and temperature of the air; (2) those resulting from the occasional (originally frequent) lapses from normal operation of the plants, due to carelessness, interruptions, changes, etc.

The results of the litigation have been generally favorable. No charters or certificates of authority to do business in New York State have been forfeited, or even suspended. The immediate effect of the commencement of the proceedings was to diminish very considerably the frequency and severity of the spasmodic or occasional odors of the second class. In addition, the defendant corporations which were the worst offenders, have made extensive efforts toward abatement by experimentally introducing various means of preventing, absorbing, and neutralizing the fumes which cause offense in New York City. The results of these efforts, which are still being continued, have even thus far been fairly effective.

The hearings and investigations by the State Commissioner of Health are still continued at infrequent intervals, mainly for the purpose of determining the progress in abatement accomplished by the defendant corporations at the plants, and of giving them further time in which to perfect their improvements. The West End Association, as well as the City of New York, maintains a vigilant and stimulating watchfulness of all proceedings and developments, and as plaintiffs are represented of course by counsel at all hearings and conferences. Thus, sanitary science has acquired another legal weapon with which to combat the nuisances arising from "some inter-state odors".

It will surely not be deemed inappropriate at this time to pay a tribute to a former Deputy Corporation Counsel who originally represented New York City as one of the plaintiffs in this case, the late Dr. W. J. O'Sullivan, physician, chemist, lawyer, gentleman, whose high personal character and unique and diversified qualifications for the difficult task of directing litigation in many highly technical matters, made him a remarkable legal character. His habitual thoroughness in preparation and faithful adherence to duty led him in the initial stages of this case to expose himself to injury from nauseating fumes, which injury was the primary though indirect cause of his ultimate untimely death. His memory as a man, a technician, and an attorney will be held in high esteem and warm affection by those who knew him intimately.

This form of legal remedy for injuries caused within a State by extra-State corporations is believed to be novel, and the present case of the West End Association and New York City against the New Jersey corporations is the first brought under the new law of 1917. It would seem that, with proper State legislation, this remedy should be applicable in other States and to other inter-state wrongs than odor nuisances, namely, to the cases of the diversion, to the interruption of flow, or to the pollution, of streams flowing through or past one State into or past another State, except in the case of

streams which are debarred from State jurisdiction by reason of their being Federally navigable or otherwise subject to Federal jurisdiction.

It is hardly necessary to point out that the efficacy of this form of remedy, as far as it relates to foreign corporations, depends entirely on the degree of importance which the erring foreign corporations attach to the privilege of doing business in the State of which the complainants are citizens.

ANDREW J. PROVOST,* JR., ESQ. (by letter).†—The author has performed a real service in calling professional attention to the obstacles in the presentation before the Courts of specific, conclusive scientific evidence regarding odor nuisances.

In other nuisance evidence, such as relates to liquid wastes, the camera, volumetric measurement, laboratory analysis, etc., can be counted on to support and to lay the foundation for an expert opinion. In regard to the smoke nuisance, also, various supporting data are obtainable. On the questions of odors, however, the expert is unable to justify his opinion otherwise than by his sense of smell, which, to the Court, must appear little different from that of the lay witnesses.

If the expert could state that he was familiar with the process of the plant, the chemical combination of the gases discharged, and the constituents thereof which produce odors; that he had isolated from the atmosphere at certain distances from his plant odor-producing elements and had subjected these to analysis according to standard technique and had identified them as the same as those produced in the defendant's plant, and that he knew from observation and study that these elements are injurious or non-injurious, as the case may be, to the health and comfort of normal human beings, how different would be his position before the Courts. Possibly he will never be able to do this, but he can, perhaps, accomplish more for human comfort by working within the plant, by perfecting methods such as have been described by Professor Whipple, whereby the odors produced in the offensive trades will be absorbed, neutralized, or destroyed before reaching the atmosphere.

Some of the problems thus far studied would include: Garbage reduction works, slaughter-houses, piggeries, fish-rendering works, sewage treatment works, etc.

Garbage Reduction Works.—Many engineers began their experiences with the Barren Island plant, referred to by the author, and have appeared as friendly advisers or as hostile critics to its managers, at one time or another, during the past thirty years. It seems unlikely that another plant on such a scale will again be attempted and attentions and studies are directed to other types of plant, to better methods of operation, and to the evolution of more efficient operatives.

Slaughter-Houses.—The establishment of large abattoirs in the thickly populated centers, has resulted in intensified effort on the part of many operators to avoid the production and escape of odors. In some instances, chemicals which are quite effective in neutralizing certain peculiarly offensive

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† Received by the Secretary, November 26th, 1921.

odors, have been tried experimentally by the writer for the neutralization of other types of odors without much success.

Piggeries.—Offensive piggeries still exist, although it has been proven that the fault lies with the operator and not with the hogs. During the World War, many of the worst types of such plants were built, which defied regulation. Injunction proceedings were resorted to in some cases, but the general need for pork and the fact that the hogs consumed military camp garbage, secured immunity from successful prosecution.

Fish-Rendering Works.—Numerous large establishments exist, usually in quite isolated locations, for the extraction of oil from fish. The waste products consist of a slimy, liquid gurry and a comparatively dry scrap. The scrap is used as a fertilizer base and, if immediately treated with sulphuric acid to fix the ammonias, it is not particularly offensive, even if stored for considerable periods in large quantities. The gurry which cannot be discharged into streams without visible nuisance, is usually treated in tanks to precipitate the grosser organic solids. Fermentation results in an odor indescribably offensive. As the result of complaint and State action in some cases, condensers have been erected, which are effective in disposing of gurry wastes without the escape of odors and, at the same time, have conserved an appreciable part of the available nitrogen. A plant of this kind, which was threatened with an order to suspend operations, was modified by the writer to such an extent that complaint was withdrawn.

Sewage Treatment Works.—The cause for odors in and about a sewage treatment plant usually arises from ignorant, incompetent, or careless operation. There are, however, certain odor-producing processes which can be largely controlled by the designer. It is known that the odors which travel farthest and which cause the most complaint are produced by spraying the effluents from sedimentation tanks in the operation of open sprinkling filters and by the open lagooning or drying of sewage sludge. In certain cases of this kind, nuisance odors persist to distances as great as ½ mile or more. The writer has been able to control these odors so as practically to avoid all complaint, even in plants located in the midst of settled communities, by housing the sprinkling filters and the sludge bed areas. The gases are then either re-absorbed by the effluent liquid or are discharged in such uniform, small quantities as to be substantially unappreciable.

Engineers who design apparatus for handling and disposing of offensive products and wastes are justified in demanding the highest efficiency in operation and regular supervision of the plant.

Odors travel usually in the direction of the wind and are frequently unnoticeable in other directions. In the absence of wind and vertical atmospheric currents, with a high barometer and with fog or mist, there appears to be a tendency for the odor-carrying gases or vapors to combine with the atmospheric moisture and to travel with it, close to the ground, until gradually released.

The plea made by the author for regulation of offensive trades by imposing restrictions on the installation of equipment and proposed methods of

operation, instead of waiting to abate a nuisance after it is created, appears to be sound and deserves careful consideration.

Research along many lines is required before sufficient knowledge of handling putrefactive and fume-producing products is acquired to permit experts to go before the Courts and satisfy them beyond a reasonable doubt that as planned a certain project will or will not create a public nuisance.

In cases where the nuisance has been created, the Courts have held, as cited by W. H. Dittoe, M. Am. Soc. C. E., in his monograph, "How to Control Nuisances from Offensive Trades", that:

"It is no defense to an indictment for maintaining such a nuisance, that the business, trade, or occupation which occasions it is a useful one, or that it is really a public benefit, contributing largely to the enhancement of the wealth, prosperity, or commercial importance of the community, or that it furnishes, on the whole, a convenience to the public which more than counterbalances the detriment it occasions. For if it is in reality a nuisance or operates as such on the public, no measure of necessity, usefulness, or public benefit will afford a justification for maintaining it. Nor is it any defense to show that the business is carried on in the most prudent and careful manner possible; that the most approved appliances known to science have been adopted to prevent injury. The question of care is not an element in this class of wrongs; it is merely a question of results, and the fact that injurious results proceed from the business, under such circumstances, would have a tendency to show the business a nuisance *per se,* rather than to operate as an excuse or defense, and the Courts would feel compelled to say that, under such circumstances, the business is intolerable, except when so far removed from residences and places of business as to be beyond the power of visiting its ill results on individuals or the public."

No doubt there are special and unusual cases of odor nuisance which are entitled to a somewhat different viewpoint. The writer has in mind some experiences resulting from his service as sanitary expert during the construction of the Catskill Aqueduct. For a distance of about ten miles, this work ran through the Croton water-shed, which at that time was furnishing most of the water supply for the City of New York. Several thousand workmen were employed, most of whom were housed in regulated labor camps. When it was contemplated to commence this part of the work, the decision was reached that all human wastes must be kept from the streams and water-courses and that the most effective means would be secured by incineration. Removable, water-tight receptacles were provided for the entire line of the work. These were collected daily and their contents burned in various types of furnaces. The strange, original character of odor produced was at the first experience quite startling. The gases, due to their high temperatures, were very volatile and were most noticeable at considerable distances and usually on higher ground. By carefully selecting the sites of the furnaces, it was generally possible to minimize the odor nuisance, and although the practice of night soil incineration was largely extended to the other sections of the work, throughout a length of about 100 miles, very little complaint was made. Possibly, this was because people felt the annoyance was only temporary and that its presence was justified largely by the necessity of safeguarding the sanitary condition of the streams.

Had the complaints been more numerous and severe, it is quite possible that means might have been provided for reducing the odors by smoke consumers of suitable type. During the five years' occupancy of the Croton watershed not a case of typhoid developed among the force employed, nor at any time was there any suggestion made that the sanitary quality of the water supply of the city had been impaired.

If injunction had been sought against the odors from these incinerators, it appears highly probable that neither the Courts nor any expert board would have felt that the alleged discomfort caused by their operation was paramount to the necessity of safe-guarding the water supply of a great city.

Garbage and other putrefactive organic wastes must of necessity be destroyed in the most rapid manner practicable, and this necessity imposes the obligation to devise apparatus and methods of operation which will avoid, so far as possible, cause of offense and discomfort to individuals and the public.

ROBERT SPURR WESTON,* M. AM. SOC. C. E.—The speaker will confine himself to a discussion of three odors: First, those given off by sugar-house waste; second, those given off by wastes from the manufacture of lactic acid; and, third, those produced by oil refineries.

In the manufacture of sugar, cane juice is expressed; the water is evaporated from it in multiple effects and vacuum pans; and cane sugar is obtained from the concentrate by crystallization. The distillate which contains considerable sugar, is usually wasted, together with some organic matter and some press-cake water containing organic matter, including various gums and sugar. Cuban sugar operations are conducted in the dry season and the wastes are discharged into lagoons or dry arroyos. These wastes begin to ferment even before they leave the sugar-house, which results in a vile odor from the hydrogen sulphide mixed with acid and the organic compounds containing sulphur; there is no odor much more disagreeable. Starch factory waste is nearest in the odors given off; but most of the starch factories, are located in cooler climates, and the nuisance is not so great. Those who have been through Cuba have experienced this odor from sugar-house wastes, and have noted the effect of the waste on the streams. The remedy for this odor lies in a better method of waste disposal, which has not yet been attained.

Lactic acid is made by hydrolyzing starch and fermenting the product. The principal source of starch is vegetable ivory, a dense tuber from Africa, which ferments, producing lactic acid. This acid soon accumulates, and inhibits the growth of the bacteria of fermentation. Fermentation starts again after lime is added. In a short time, the process reaches its limit, and the lime and organic matter are precipitated. The precipitate is removed by filtration, and the filtrate containing the lactic acid is concentrated in vacuum pans. If the various wastes from this process are discharged into a lagoon or a stream where dilution is insufficient, they ferment with the production of butyric and other organic acids, hydrogen sulphide, and also compounds of sulphur and organic matter. Between New York City and Boston, Mass., a

* Boston, Mass.

plant was started during the war, and continued until it was put out of operation in 1919 by the action of a town board of health. The odor from the untreated waste was observed at times $2\frac{1}{2}$ miles away and white lead paint within a radius of $\frac{1}{4}$ mile was turned black by the sulphur gases given off.

During the last year the speaker has dealt with the odors produced by oil refineries, which odors Mr. Gage has also described. The source of fuel oil, is crude petroleum, and refineries are using more and more Mexican oil which is rich in sulphur. This oil is split by distillation into fractions, the lighter fractions being kerosene and gasolene. A refinery sometimes produces lubricating oils, gas oils, fuel oil, and, in addition, the heavier products, such as paraffin, asphalt, and coke. For many generations the people of Massachusetts have been accustomed to the odors produced by the drying of the descendants of the "sacred codfish"; but oil odors are new and terrifying. Not only were objections made to the odors from the newly constructed refineries, but the smoke produced by the burning of oil was also a cause of complaint. Coal furnaces which had been hastily altered to burn oil, frequently produced a sulphurous smoke, particularly on Monday morning when late firemen tried to make up for lost time. Where the vapors and smoke were uncontrolled, conditions became so that people closed their windows at night to shut out the bad smells, and cases of nausea and vomiting were common.

Economic conditions demanded oil for fuel, and more refineries were completed. Locations were often unwisely chosen, and neighboring districts were gassed until the residents rebelled. This intolerable condition resulted in a demand for the cessation of either the odor or of oil refining. Certain refineries have stopped odor production; others have been forbidden to perform certain offensive operations, such as making coke, etc.

The means taken by certain refineries to control the production of odors have not been successful, their failures having been due to poor engineering, and especially to poor management, rather than to inherent difficulties. The most successful plants have either put all the refining apparatus, including tanks, under a vacuum system, and have passed all the gases through a fire, or subjected them to treatment by washing with some of the raw material going through the process; that is, the counter-current principle has been applied by washing the distilled gases with the raw material about to be distilled.

The washing of gases in water simply transfers the odors to the water which has at least as much tendency to give off gases to the air as the oil itself, and very often the discharged wash water simply transfers the odor to another location.

In observing an odor-producing plant, the first effort should be to make the adjectives which have been used so generally to describe odors, to mean something. One way of doing this is to have the smelling systematic, that is, locate the odors from the leeward side of the plant, in order to determine the distance an odor can be detected under various air conditions. By plotting the observations day after day, one can obtain a graphical representation of the characteristics of any plant for odor production.

In order to make the record of one gas more complete, the speaker's firm, with the aid of the Wallace and Tiernan Company, Incorporated, adapted what is known as the Palmer dust apparatus for measuring the hydrogen sulphide in the air. The method was more delicate than the human nose, which, ordinarily, was able to detect about 1 part of hydrogen sulphide in 10 000 000 parts of air. This is a low concentration, but with the apparatus referred to, as little as 15 parts of hydrogen sulphide in 1 000 000 000 parts of air could be detected or about one-eight of the quantity which the observers could smell.

This Palmer spraying apparatus is really a nebulizer, which absorbs gases passed through a nebulized absorbent. It consists of a U-tube with a condenser, through which air is drawn by a fan and connected with the fan is a Venturi meter. It was customary to draw 100 liters of air through this device in 4 min.; 100 liters is about equal to $\frac{1}{2}$ bbl., so that a considerable sample of air was tested at one time. By having three assistants cover the 24-hour period, an accurate record of the hydrogen sulphide content was made. In the tests, 35 cu. cm. of $\frac{\text{Normal}}{10}$ caustic soda was put in the U-tube. The air was then drawn through the liquid which nebulized and absorbed the gas. The caustic soda solution was then washed out of the U-tube and the hydrogen sulphide determined by titration with $\frac{\text{Normal}}{1\,000}$ iodine solution, using starch for an indicator.

This apparatus was tested thoroughly, and it was found that it would absorb all the hydrogen sulphide from the air passed through it and also any sulphur dioxide. Furthermore, it had an accuracy of $\pm$ 15 parts in 1 000 000 000.

It is well that engineers are attacking this odor problem. Sanitary engineering was a combination of bacteriology, chemistry, climatology, hygiene, and civil engineering. This new work which the speaker hopes will not be given a new engineering name, includes also meteorology, physics, and chemical engineering. The concentration of population and the rising esthetic standards will make work for engineers in these new fields.

Alexander Potter,* Assoc. M. Am. Soc. C. E.—A few years ago, the speaker was asked to investigate the existence of odors in certain sewers. In the course of the investigation, the question of the travel of odors through sewers was studied and some interesting results were obtained both as to their direction and rate of travel. It was found that offensive odors were given off thousands of feet down stream from the point of emanation.

About twenty years ago, the speaker constructed a system of trunk sewers for a number of municipalities in New Jersey, including portions of the Cities of Elizabeth, Newark, Summit, Irvington, South Orange, West Orange, Millburn, and other places. During the construction of this system of trunk sewers, he also had supervision of the construction of most of the lateral systems within the confines of the various municipalities. Flush tanks were recommended and constructed throughout most of the lateral systems, but the main

* New York City.

house-trap was omitted through most of the district, ventilation taking place through the house-risers.

In only one municipality did the local health authorities insist on the use of the house-trap, but, in order to economize on water, flush tanks were not put in service, dependence for flushing being placed on the use of a hose stream in the upper end of the sewers. As often happens under such conditions, the sewers were not flushed for weeks and months at a time.

Although the conditions as to capacity, flows, grades, and velocities were similar throughout the entire district, complaints were made from time to time about odors along the Joint Trunk Sewer through this municipality, which was built at joint expense to provide sewerage facilities for certain outlying districts.

Repeated examinations of the Joint Trunk Sewer disclosed nothing that would justify the conclusion that the odors in this section of the trunk sewer had their origin therein.

The speaker's contention was that the odors were caused by unsanitary conditions of the lateral sewers, from which odors found their way down grade into the Joint Trunk Sewer. The air space above the flowing sewage in the Joint Trunk Sewer was proportionately very much less than the combined air space in the many contributing laterals; therefore, as the foul air found its way down stream to the Joint Trunk Sewer, it was forced out through the manholes, the quantity fluctuating with the volume of sewage during the different hours of the day.

It is a popular belief that the direction of the flow of sewer air is up hill. Experiments were made to substantiate the theory that the flow was down hill. At certain points along the Joint Trunk Sewer and the lateral sewers, smoke balls were introduced and the direction of the smoke was noted. The velocity of the smoke and its volume passing down the sewer, together with the velocity of the flow of the sewage, is indicated in Table 3.

TABLE 3.—Summary of Experiments to Determine Movement of Air in Sewers, October, 1916.

No. of experiments.	Sewer:		Average Velocity:		Ratio, in percentage.	Movement of air, down stream, in cubic feet per day.
	Size, in inches.	Grade, per hundred.	Sewage, in feet per second.	Air movement, in feet per second.		
1	8	1.9	1.6	0.32	20	20 300
2	12	0.25	3.2	0.24	8	12 100
3	10	2.3	4.0	1.50	35	60 100
4	10	0.84	3.4	1.20	35	125 000
5	10	0.84	3.4	1.12	33	121 000
6	10	6.5	2.7	2.38	88	89 700
7	15*	0.56*	3.1 main line	1.94	63	175 000
			3.0 relief	0.29	10	41 600
8	18-20	1.56-0.28	4.3	1.06	25	112 000
9	12	0.75	3.1	1.30	42	58 300
10	15	0.40	1.82	0.29	16	26 800

*Main line only.

The speaker believes that the controlling factor in directing the flow of the air down stream is the motion imparted by friction with the flowing sewage.

Down-stream components are thus produced sufficient to carry the air in the direction opposite to that which might reasonably be expected.

Table 3 records velocities as high as 2.38 ft. per sec., which were sustained for distances of ½ mile in certain sections of the sewer.

As a result of these experiments, the speaker not only succeeded in having the sewers cleaned more frequently, but also had the traps eliminated so that gases forming in the sewers were freely vented to the atmosphere at each house connection, and thus the nuisance was abated.

CALEB MILLS SAVILLE,* M. AM. SOC. C. E. (by letter).†—As a boy the writer well remembers the smell and the choking sensation, experienced at night, when a strong east wind blew up the river valley in which his home was located. This wind was followed in the morning by a yellow haze, which left a dark discoloration on the houses in the lower part of the town.

The conditions producing the odor mentioned were due to emanations from a chemical manufacturing plant about 4 miles to the east, of which sulphur products were the principal industry.

Subsequently, in another city, a large gas plant belched out its fumes at night to such an extent that it was impossible to sleep, at a distance of 4 or 5 miles away, on certain nights when wind and other atmospheric conditions were in conjunction. Happily for the inhabitants who are obliged to live in these places, both these nuisances no longer exist. Their disappearance, however, was not due so much to the regard of the plant directors for the comfort of the neighboring communities, as it was to the results of experimentation and advance in chemical science, which results indicated that valuable products were being wasted. Perhaps, a solution of the problem of community preservation from nauseating odors, would be to put industrial chemists at work to find a process which will dangle the lure of the dollar before the eyes of the plant manager. The possibility of such attainment might cause him to seek eagerly to conserve his filthy smells in order to exchange their cause for money. Possibly, we have not yet advanced far enough along scientific lines to cope successfully with many stages of this subject, and it may be that later developments will disclose the fact that all these odors are as yet unrecognized indications of a present waste which, when properly treated and conserved, will yield products too valuable to be overlooked. For example, it was only a comparatively short time ago that the gases of combustion were wasted, yet, although people were susceptible to the coal-gas smell which was more or less noisome to some, it was not recognized until recently what waste was going on in unconsumed fuel.

The author remarks on the hopelessness of attempts at the mathematical measurement of odors and rightly mentions the disagreement as to the character and intensity of the same odor. Although this is true, as applied to those who are untrained, it is equally true that the organs of taste and smell are susceptible of marvellous development. The practical agreement of several trained observers as to the intensity of odor of a water and the almost uncanny

* Hartford, Conn.

† Received by the Secretary, November 14th, 1921.

unanimity of opinion of several tasters of coffee, tea, and spice is sufficient indication of what it is possible to attain.

In this connection, also, the writer would like to call attention to the unique and valuable work of George C. Whipple, M. Am. Soc. C. E., in classifying the comparatively faint odors of potable water. On page 18 of his book on the "Value of Pure Water", Professor Whipple gives a diagram from which he says "one may calculate what may be called the esthetic deficiency of water". The curves of this diagram are built up "by adding together the percentages of objecting consumers", 100% being an odor to which everybody would object. In his discussion, Professor Whipple recognizes the difficulties of classifying odors and gives three separate equations for satisfying as many conditions.

Although these curves relate, as stated, to the comparatively faint odors of a potable water supply, it may be that a similar classification can be worked out for the odors of various industries. After some study, a scale may be arrived at, whereby it will be practicable to make more definite statements as to the probable number of people in the area exposed, who will be affected seriously by a certain odor, and thus decide whether or not the odor is a nuisance. It is well known that some persons are especially sensitive even to slight odors; however, if only a small percentage of the whole number are affected, the condition could hardly be called a public nuisance and a subject for drastic action.

Whatever conclusions are arrived at, in any individual case, it must be recognized that the solution of the problem lies in a proper balance of many conflicting elements, and that, very likely, the matter may resolve itself into a decision as to the degree of odor permissible rather than total elimination. The standard probably would be based on the reasonable use of the air, the possibilities of practical remedial action, and the economic value of odor elimination, as opposed to air pollution by the discharge of noxious or ill-smelling vapors.

Proceeding along this, or a similar, line, it would seem that after a time a sufficient number of precedents would be established so that it would be possible with some degree of certainty to bring forward such evidence of polluted air and its results, as affecting persons and property, that the Courts would be willing to apply measures for relief. For this purpose, however, trained observers are required, for the ordinary individual is incapable of differentiating as to intensity of odor and, indeed, it is probable that a large proportion of the people of a community are unable, quickly, to determine whether it is taste or smell which offends them.

Proceeding along the lines which have been suggested, it is probable that a valuation curve, or rather one of depreciation could be worked out similar to Professor Whipple's valuation of attractive water, from which the financial loss to a community, as well as the personal annoyance of the inhabitants, could be determined. Such evidence, if it were available, would be much more effective in abating a nuisance than many individual complaints of a more or less indefinite nature, which when investigated become often of little value, because of differences in the personal equation of the observer.

In general, the matter of elimination of disagreeable odors seems to be strictly analogous to the pollution of natural water-courses into which sewage and waste are discharged by the method of disposal by dilution. Some of the methods found to be helpful in correcting unbearable conditions in regard to water pollution may, perhaps, be useful in dealing with the pollution of the air.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

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THE FLOOD OF JUNE, 1921, IN THE ARKANSAS RIVER, AT PUEBLO, COLORADO

Discussion*

BY MESSRS. ROBERT FOLLANSBEE AND E. E. JONES.

ROBERT FOLLANSBEE,† M. AM. SOC. C. E., and E. E. JONES,† ESQ. (by letter).‡—The U. S. Geological Survey has recently completed an investigation of the Arkansas River flood for the purpose of publishing a water supply paper on its hydrologic features. The field work was performed by one of the writers, Mr. Jones, and the office studies and report were prepared by Mr. Follansbee. The following discussion is based on this investigation.

The small areas of intense rainfall and the rapid rise of the river at Pueblo, and almost as rapid fall, were the remarkable features of the Arkansas River flood of June, 1921. Unfortunately, no regular records are available for the areas of heaviest rainfall and an idea of the precipitation can be gathered only from statements of local residents of those areas. A summary of such statements obtained by personal interviews is shown in Table 22.

The rain of June 3d began about 1 P. M. in the foothills north of the valley. By 3 P. M., it had spread over the upper and middle parts of the valley and between 5 and 7 P. M. had reached the lower end near Pueblo. The hardest rain, on Eightmile, Rush, and Rock Creeks, occurred between 3 P. M. and 4 P. M., while near Pueblo it did not occur until 10 P. M. or 11 P. M. The rain continued with intermissions until sometime after midnight.

For the 48 hours ending on the afternoon of June 4th, the Weather Bureau records indicate that the general rainfall in the Arkansas drainage basin between Canyon City and Pueblo had been from 3 to 5 in., being heaviest in the northern part of the area near Pike's Peak.§

* Discussion of the paper by James Munn and J. L. Savage, Members, Am. Soc. C. E., continued from November, 1921, *Proceedings*.

† Denver, Colo.

‡ Received by the Secretary, November 25th, 1921.

§ An investigation of the Colorado Springs records indicates that an error was made in measuring the rainfall at that point, the original figures being 2¼ times too great. As corrected, the rainfall from June 2d to June 6th, was 5.18 in., which agrees closely with 5.91 in. at Lake Moraine and 4.03 in. at Victor, both near-by stations.

Two principal areas of intense rainfall are indicated from the statements of local residents and the maximum discharges of the tributary streams. The larger area extended from a point a few miles east of Wigwam near the southern boundary of El Paso County to the top of the Wet Mountains near Beulah, a distance of 45 miles, and from a point a short distance above the mouth of Rush Creek nearly to Pueblo, a distance of 15 miles. The smaller area, covering the southern slope of the Pike's Peak uplift which forms the northern part of the valley, extended from a point above the Skaguay Reservoir to a point 3 or 4 miles south of the river, a distance of 25 miles, and from Oil Creek to Beaver Creek, a distance of 11 miles. The combined areas cover about 700 sq. miles.

An investigation was made of the tributary streams between Canyon City and St. Charles River, a few miles below Pueblo, to determine the maximum discharge of each stream and the approximate time of the discharge. In addition, the maximum discharge of the Arkansas at Pueblo was estimated. The work began on July 6th, 1921, and the high-water marks left by the flood in the various streams could still be readily detected.

The method used was to select on each tributary stream a portion as near the mouth as possible where the maximum discharge had been confined to the channel, and where the channel was sufficiently straight to eliminate as far as possible the effect of the water piling up on the outer side of bends. Cross-sections were selected at the upper, middle, and lower end in each part of stream measured, and stakes were set at the high-water line on each side of the channel. Beginning at the up-stream end, a careful level line was run around the six stakes, closing on the initial stake. Distances on each side between stakes were measured along the high-water line. Between each pair of stakes on opposite sides of the channel the cross-section was carefully measured. Owing to the effect of bends in the channel, the difference in elevation between adjacent cross-sections was not the same, nor was the distance along the high-water line the same. Both distances and differences in elevation between adjacent cross-sections on opposite sides of the channel were averaged to determine the slope between them.

Each cross-section was plotted on a large scale and the area determined by a planimeter. The discharge between the upper and middle sections, and the middle and lower sections was computed by Kutter's formula, using the average area of the two sections. The discharge through all three sections was also computed, using the total slope and the average of the hydraulic factors for all sections. Where the difference in area between the upper and lower cross-sections made a marked difference in the velocity through each section, a correction for velocity of approach was applied to the slope between them. Although the conditions of flow were not the same for all sections, the main channels were uniformly free from vegetation, and as the overflow areas were only a small percentage of the cross-section, a uniform value for n of 0.035 was used for the tributary streams.

In Table 23 are shown the results of this investigation for each tributary stream between Canyon City and Pueblo.

TABLE 22.

Location.	Statement regarding rainfall.	Duration of Rainfall: Began.	Duration of Rainfall: Ended.
North of Arkansas River.			
Dry Creek near mouth	Cloudburst	4 P. M.	....
Dry Creek, Sec. 26, T. 20 N., R. 65 W	Hardest at 10 P. M	7.30 P. M.	....
Sec. 27, T. 20 S., R. 65 W., just west of Pueblo	12 in. (measured in concrete box)	Night of June 3 4.	....
Teller Reservoir on Turkey Creek	10 in	3 P. M.	Morning, June 4.
Skaguay Reservoir on West Beaver Creek	7.5 in. (measured in bucket, morning, June 5)	....	....
Three miles east of Penrose	7 in. (measured in standard rain gauge); hardest at 9 P. M.	3 P. M.	8 A. M., June 4.
Penrose	....	7 P. M.	Morning, June 4
Brush Hollow Creek in Sec. 13, T. 19 S., R. 69 W	....	8 P. M.	Midnight.
Eightmile Creek, 5 miles above mouth	10 in. (hardest about 3.15 P. M)	3 P. M.	....
Eightmile Creek in Sec. 15, T. 19 S., R. 69 W	....	5 P. M.	11 P. M.
Oil Creek in Sec. 35, T. 18 S., R. 70 W	No cloudburst; ordinary hard rain	....	....
South of Arkansas River.			
Chandler Creek ½ mile west of Florence	9 in. (measured in bucket); did not extend up Chandler Creek 3 miles	Hardest at 6.30 P. M.	....
Hardscrabble Creek, divide southeast of Florence	4 in., water ran over prairie	3 P. M.	....
Rush Creek in Sec. 9, T. 21 S., R. 67 W	....	3 P. M.	11 P. M.
Rush Creek in Sec. 22, T. 20 S., R. 67 W	Cloudburst for 30 min.; hardest rain 2 miles south	....	....
Peck Creek, 8 miles above mouth	Tremendous rain; water ran everywhere	2.30 P. M.	....
Peck Creek in Sec. 34, T. 21 S., R. 67 W	....	4 P. M.	....
Cameron and Osteen Arroyos in Sec. 30, T. 20 S., R. 66 W	Five periods of very hard rain	3 P. M.	Midnight.
Head of Rock Creek in Sec. 34, T. 21 S., R. 67 W	4 in.; water on level	4 P. M. 9 P. M.	6.30 P. M. Midnight.
Between Rock and Soda Creeks, Sec. 29, T. 21 S., R. 66 W	Horse drowned in open field	3-4 P. M.	....
Boggs Flat, about Sec. 35, T. 21 S., R. 66 W	5 in. in 30 min.; 6 in., water ran over prairie	5 P. M.	Morning, June 4.
Blue Ribbon Creek in Sec. 32, T. 20 S., R. 65 W	Hardest at 10 P. M	4 P. M.	....
Blue Ribbon Creek in Sec. 2, T. 21 S., R. 65 W	10 in. (hardest at 10 P. M.)	3 P. M.	....
Beulah in Sec. 3, T. 23 S., R. 68 W	Hard rain all night	6 P. M.	4 A. M., June 4.

NOTE.—Except where the method of measuring rainfall is stated, the quantities can be considered only as approximate, as the observer had no means of making an accurate measurement.

TABLE 23.—MAXIMUM DISCHARGE OF TRIBUTARY STREAMS BETWEEN CANYON CITY AND PUEBLO.

Stream.	Distance above Union Avenue Bridge, Pueblo, in miles.	Tributary from north or south.	MAXIMUM DISCHARGE, IN SECOND-FEET:		Drainage area, in square miles.	Time of flood crest.
			Total.	Per square mile.		
Dry Creek	1.8	N.	24 400	283	86	11 P. M., June 2.
Blue Ribbon Creek	4.2	S.	9 130	1 360	6.7	11 P. M., June 3.
Arroyo	4.8	S.	1 910	1 060	1.8	11 P. M., June 3.
Boggs Creek	8.2	S.	15 400	582	26.5	6.30 P. M., June 3.
Arroyo	9.5	N.	9 740	619	15.8	5 45 P. M., June 3.
Rock Creek	9.8	S.	53 900	913	59	9 P. M., June 3.
Peck Creek	11.0	S.	19 400	564	34.4	5 P. M. or 6 P. M., June 3.
Cameron Arroyo	12.0	S.	13 900	1 900	7.3	9 P. M., June 3.
Osteen Arroyo	13.5	S.	9 060	1 160	7.8	9 P. M., June 3.
Turkey Creek*	14.2	N.	9 000	188	48	8.30 P. M., June 3.
Rush Creek	16.8	S.	4 670	238	19.6	5.30 P. M., June 3.
Red Creek	19.8	S.	911	22	40.6	5 P. M., June 3.
Fred Rohr Gulch	20.0	N.	968	104	9.3	5 P. M., June 3.
Ritchie Gulch	22.0	S.	920	41	22.6	5 P. M., June 3.
Beaver Creek†	25.0	N.	9 470	45	213	7.30 A. M., June 4.
Hardscrabble Creek	28.8	S.	3 300	19	173	7.30 P. M., June 3.
Brush Hollow Creek	31.0	N.	5 320	243	21.9	8 P. M., June 3.
Eightmile Creek	33.5	N.	10 000	154	65	8.30 P. M., June 3.
Coal Creek	34.5	S.	3 720	167	22.3	7 P. M., June 3.
Oak Creek	35.2	S.	2 760	41	68	7 P. M., June 3.
Sixmile Creek	35.8	N.	1 890	77	24.6	7.30 P. M., June 3.
Chandler Creek	36.2	S.	1 610	118	13.6	7 P. M., June 3.
Oil Creek	40.2	N.	2 510	6	425	11 P. M., June 3.

* Forty-eight square miles is the area of Turkey Creek water-shed below the Teller Reservoir, as little or no water passed the dam during the period of maximum discharge of the creek.

† The discharge given for Beaver Creek is the maximum due to the storm which occurred more than 24 hours before the failure of the Schaeffer Reservoir (9 miles above the mouth of Beaver Creek). At the time of the maximum natural discharge of Beaver Creek the quantity passing the spillway and waste-gates of Schaeffer Reservoir was computed to be 3 100 sec-ft. To this was added the drainage to Beaver Creek from 68.5 sq. miles between the dam and the mouth. The maximum discharge of Red Creek, the chief tributary, was found to be 4 490 sec-ft., or 93 sec-ft. per sq. mile. Applying this unit run-off to the entire additional drainage area gives a run-off of 6 370 sec-ft., which added to the 3 100 sec-ft. passing the dam, gives a total maximum natural discharge of 9 470 sec-ft. for Beaver Creek at its mouth.

A study of each tributary showed that the first flood crests to reach Pueblo were those from Boggs, Peck, Rush, and Red Creeks, and Fred Rohr and Ritchie Gulches, all within a distance of 15 miles, and that the time of their arrival was between 7 P. M. and 8 P. M. Although the heavy rains continued during the afternoon and evening and a considerable volume of water entered the river, no more flood crests reached Pueblo until late in the evening. From 10 P. M. until midnight, the floods from the tributaries near Pueblo, which were the streams having the greatest flood discharge, reached the city. At the same time the flood crests from the most distant tributaries arrived, which caused the rapid rise in the river at that time. The period of intense rainfall was so short and the slopes of the tributary streams so steep that the period of crest discharge was short. This is shown by the rapidity with which the river fell after reaching its peak.

The maximum discharge at Pueblo was determined by measuring the high-water cross-section and slope at a point just west of Pueblo and above Dry Creek. The discharge was found to be 83 500 sec-ft. (using $n = 0.030$), and the discharge of Dry Creek at the time of the maximum stage of the

Arkansas was estimated as 19 500 sec-ft., or a total of 103 000 sec-ft. as the peak flow of the river at Pueblo. From a hydrograph of the river at that place, based on records compiled by engineers of the Denver and Rio Grande Western Railroad Company and data used by the authors, the total discharge at Pueblo from 8 A. M., June 2d, to midnight of June 5th, was found to be 145 000 acre-ft. Of this quantity, 90 000 acre-ft. was the total flow from noon of June 3d to midnight of June 4th.

The nearness to Pueblo of the areas of intense precipitation, and the fact that the flood crests from the streams nearest Pueblo arrived at the same time as the crests from the more distant tributaries, combined with their unprecedented unit run-offs, made a condition that could only result in a flood of great magnitude.

The maximum discharge of the flood of 1894 was only 40% of the maximum of the recent flood and caused by rains extending over the entire drainage basin. At the time of the 1894 flood, the *Rocky Mountain News* recorded that heavy precipitation on May 30th and May 31st extended over the Arkansas drainage basin, in the form of snow at the higher elevations—notably Pike's Peak and the mountains in the upper end of the basin near Leadville. On the morning of May 30th, Salida reported that rain had fallen continuously for 36 hours and probably would continue during the night. The storm at that point, for duration and volume, exceeded anything in the memory of the oldest inhabitant. The authors' Table 4 shows that at the regular rainfall stations the storm of 1894 was more severe than that of June, 1921, and yet with a storm more widespread and of greater general precipitation, the maximum discharge of the river was less than one-half that of the recent flood. During the storm of 1894 small areas of intense precipitation were apparently absent. The important part played by these small areas so near Pueblo outweighs any general deductions which may be drawn from the relation of elevation to rainfall, in its effect on possible floods in the Arkansas River.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS AND DISCUSSIONS

RAINFALL AND RUN-OFF STUDIES

Discussion*

BY MESSRS. THADDEUS MERRIMAN, R. A. HILL, KENNETH ALLEN, AND L. STANDISH HALL.

THADDEUS MERRIMAN,† M. AM. SOC. C. E.—The papers presented by Mr. Grunsky and by Messrs. Munn and Savage, illustrate two of the prevailing methods used in the study of flood magnitudes. In the first method a formula is deduced to express the maximum flood discharge in its relation to the intensity of the rainfall and the physical features of the water-shed. The second, premising an observed maximum flood discharge, recognizes the possibility of the occurrence of a still greater flood and increases the observed values in some definite proportion, as by assuming an increase in the total run-off and then working backward for the purpose of determining the probable peak discharge. The first of these lines of thought has been well set forth by Mr. Grunsky and the second by Messrs. Munn and Savage. A third method which is perhaps the one most commonly used, consists in plotting observed maximum flood discharges with respect to the area of the drainage basins on which they occurred. An expression is then determined which will enclose, and yet fairly represent, the plotted points.

Each of these three methods constitutes a perfectly legitimate way of approximating the truth, but only the second stresses the fact, which the speaker considers to be of great importance, that greater floods than any which have been observed may occur at any time. No formula or expression for flood discharge can be used implicitly as it is written. Allowance always must be made for the case which the formula does not include.

The one outstanding feature in any tabulation of recorded floods seems to be that most of them have occurred in recent years. In a tabulation which lists 274 floods in the United States, 219 have occurred since 1890, and evidently most of them are far from being possible maxima. Obviously, this does not mean that more floods have actually occurred in recent years, but is simply the result of the absence of long-time records.

* Discussion of the paper by C. E. Grunsky, M. Am. Soc. C. E., continued from November, 1921, *Proceedings*.

† New York City.

In 1914, the speaker visited the Ohio region which had been desolated by the floods of March in that year. He then became much interested in this question of probable maximum flood magnitude and has given it a great deal of thought and study. This matter is one of great moment. Our cities, day by day, are spreading wider and farther over the flood-plains of the rivers on the banks of which they were originally founded. The buildings, bridges, fences, and other structures operate to obstruct the free discharge of the flood waters and, where the flood-plain is covered, the greatest damage occurs along the line of greatest resistance. This line of greatest resistance unfortunately often lies within the city and accounts for the appalling devastation which has come to be a rather common occurrence. Immediately after such a flood, the newspapers carry large headlines describing it as "unprecedented", "of a magnitude never before known" and as "a wall of water which swept everything before it". After a brief lapse, when the toll of the dead and the missing has been taken, the occurrence is promptly forgotten. Only in those cities which were the actual sufferers do the people live in the fear of another similar occurrence.

It is not necessary to look far to note that practically all sections of the United States are subject to a flood hazard. The records of the very recent past clearly demonstrate this and within the past 20 years "unprecedented" floods have occurred in New Jersey, New York, Pennsylvania, Ohio, Indiana, Iowa, North Carolina, Tennessee, South Carolina, Texas, Colorado, Arizona, and perhaps, the most remarkable of all in regard to volume, occurred at Monterey, Mexico. The list might be extended, but emphasis would not be gained thereby.

Each new flood is "unprecedented" simply because the records are meager and cover a period too short to be of great value. The importance of a flood in this day and generation is measured by the destruction which it occasions, whereas a flood of equal magnitude, fifty or seventy years ago, might easily have passed unnoticed and unrecorded. When one of these "unprecedented" floods does occur, the newspapers, and subsequent technical reports record the failure of one or more dams due to inadequate spillway capacity. Most dams are designed with a factor of safety of about two, but very few spillways are designed with any factor of safety; in many instances, they are made merely large enough to pass the greatest flood of record in the particular vicinity of the dam, or are proportioned on the basis of a formula and no allowance is made for the occurrence of a flood greater than those on which the formula was based.

In view of the generally unsatisfactory knowledge respecting flood magnitudes, the speaker desires to urge the necessity of providing ample spillway capacity on all dams, wherever they may be built, and as a guide to the possible maximum flood, that greater dependence be placed on the observable physical features of the flood-plains of the river itself than on any records of flood discharge which are now available or will become available for many years. By the observable physical features of the flood-plains of the river itself the speaker refers to those evidences which a flood always leaves. He does not, however, allude to the ordinary high-water drift of logs, etc., which is ephemeral, but refers to those relatively permanent and definite evidences in the deposits

of sand and gravel found on the flood-plains near the points where tributary streams first emerge on the flood-plain of the main river valley. When a river is in flood and its flood-plain is covered, the tributary streams flowing at a relatively high velocity are suddenly checked when they enter the comparatively quiet water over the flood-plain. At these points their burden of sand and gravel is deposited in the form of flat topped banks or terraces, the tops of which are only a little below the high-water level in the main stream. Later, these banks may be more or less eroded as the flood subsides and the tributary flows over and scours out the material previously deposited. A study of these evidences is of the nature of a geological investigation with respect to the recent valley deposits and will disclose the heights at which flood waters in the valley have stood at some time in the past. In this manner, the records can be carried back farther than by any other method of which the speaker is aware. He has seen gravel banks of this nature on the Miami River, which were deposited during the flood of 1914, and the tops of these banks are definite and permanent marks of the height at which the flood waters stood in that year. He also has seen similar deposits in North Carolina at elevations above those reached by the great flood of 1916, which flood at certain points raised the previous maximum stage from 21 to more than 42 ft.

Those who are located within the area which was covered with ice during the glacial period have relatively few opportunities for making such studies. In these glaciated areas, it is generally very difficult to distinguish clearly between the deposits of the glacial age and those of comparatively recent formation. Therefore, the suggestion is made that other members of the Society, located in unglaciated areas, by a study of these features, may be able to add greatly to the present imperfect knowledge respecting probable maximum flood heights.

R. A. Hill,* Assoc. M. Am. Soc. C. E. (by letter).†—A method somewhat similar to that described by Mr. Grunsky was developed by the writer to determine the probable rainfall on the mountains between the Southern California valleys and the deserts which lie back of them. The topography of that section of California south of the Tehachipi Mountains is characterized by high ranges which roughly parallel the coast at a distance back of from 40 to 80 miles and which rise out of a plain generally less than 1 000 ft. above sea level. Several of the peaks are more than 10 000 ft. in elevation, although the crests of these ranges are, in general, at elevations of from 5 000 to 8 000 ft. Twenty miles across the mountains from the highly developed coastal valleys lie the Mojave and Colorado Deserts.

The average rainfall per season, in the coastal valleys, is about 16 in. On the ocean side of the mountain ranges, the precipitation increases very rapidly for increases in elevation, up to about 5 000 ft. Above that point, the rate of increase is small, and, as pointed out by the author, there is an apparent decrease for the last few hundred feet below the crest. The rainfall, including snowfall, at or near the crest, is approximately twice as great as in the valleys a few miles below. On the desert side of the crest, the rainfall decreases rapidly and at the same elevation as the coastal valleys, only about one-quarter as much rain falls in any season.

* Los Angeles, Cal.

† Received by the Secretary, November 14th, 1921.

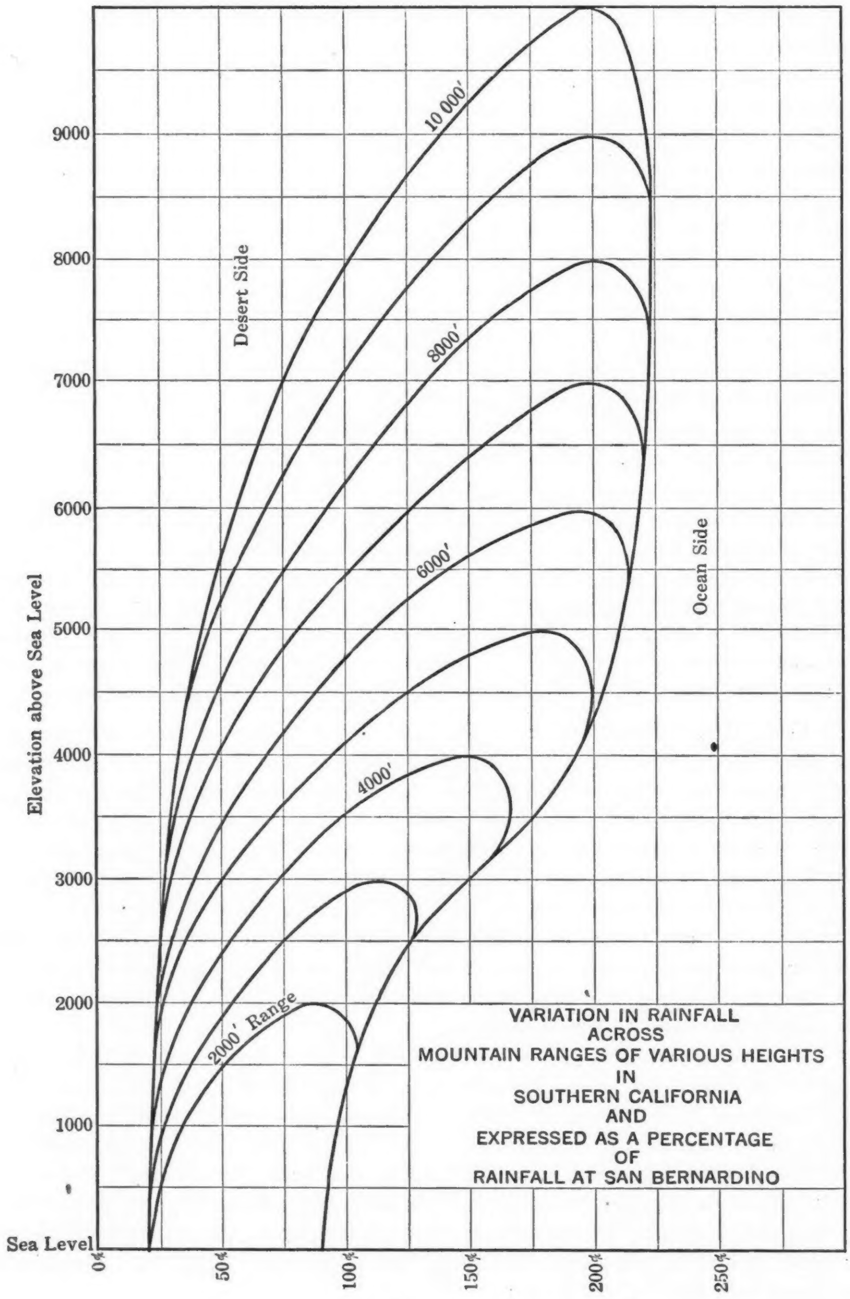
VARIATION IN RAINFALL
ACROSS
MOUNTAIN RANGES OF VARIOUS HEIGHTS
IN
SOUTHERN CALIFORNIA
AND
EXPRESSED AS A PERCENTAGE
OF
RAINFALL AT SAN BERNARDINO
Elevation above Sea Level
Desert Side
Ocean Side
10 000'
8000'
6000'
4000'
2000' Range
9000
8000
7000
6000
5000
4000
3000
2000
1000
Sea Level
0%
50%
100%
150%
200%
250%

FIG. 10.

The direction of the storms is usually from the southwest, off the Pacific Ocean. The irregular crest line and the high peaks close to the ocean side cause very definite rain shadows.

An excellent example of this is found near the top of the San Bernardino Mountains in the vicinity of Big Bear and Baldwin Lakes. The precipitation averages about 32 in. per season at the dam at the west end of Big Bear Lake (Elevation 6 500), whereas 5 miles distant at the east end of the lake the rainfall is about 21 in. per season. At Baldwin Lake, which lies about 4 miles still farther east at about the same elevation the rainfall is about 16 in. per season. This rain shadow is caused by the high peaks of the San Bernardino and San Gorgonio Mountains and the connecting ridge, all of which are over an elevation of 10 000 ft. and intercept the storms from the ocean. The west end of Big Bear Lake, however, is clear of the shadow of these higher mountains.

The scarcity of and the relatively short duration of records of precipitation gave rise to a method quite similar to that described by the author. Instead of determining the mean for the period available and comparing that with the same period at the base station to ascertain the ratio between the rainfalls at the primary and secondary stations, this ratio was determined for each season. The long record of the rainfall at San Bernardino (1870 to date) and the geographic position of that city made it desirable to refer all other stations to it.

Accordingly, with the rainfall each season at San Bernardino as a base, the rainfall for the same season at all other available stations along and back of these mountain ranges was computed as a percentage of that at San Bernardino. These points, several hundred in number, were then plotted with elevations as ordinates and percentages of the rainfall at San Bernardino as abscissas. There was considerable fluctuation in the points, as was to be expected from the widely variant conditions which exist over these mountains. Curves drawn through the weighted centers of these points gave much the same result as if the ratio between the means had been taken in the first place.

The resulting wing-shaped diagram (Fig. 10), fits quite satisfactorily the conditions known to exist on the ocean and desert sides of the mountains of Southern California. Table 13 gives the results of such comparisons for some

TABLE 13.—RAINFALL OF SOUTHERN CALIFORNIA STATIONS AS COMPARED TO SAN BERNARDINO.

Station.	Elevation, in feet.	Position relative to mountains.	Elevation at crest of mountains, in feet.	Probable percentage of rainfall at San Bernardino.	Mean rainfall for same period at San Bernardino, in inches.	Probable mean rainfall at station, in inches.	Actual mean rainfall at station, in inches.	Difference, in inches.
Barstow	2 105	Desert	8 000	25	14.98	3.74	3.62	+ 0.12
Cabazon	1 779	Desert	2 500	55	15.09	8.30	10.48	— 2.18
Mojave	2 751	Desert	6 000	35	16.14	5.65	5.00	+ 0.65
Palm Spring	584	Desert	10 000	20	15.42	3.09	4.13	— 1.04
Tehachipi	3 964	Desert	5 500	75	16.14	12.10	10.61	+ 1.49
Mt. Wilson	5 850	Crest	5 850	195	14.43	28.20	25.20	+ 3.00
Bear Valley Dam	6 500	Ocean	8 000	220	14.71	32.40	32.74	— 0.34
Glen Ranch	3 258	Ocean	9 000	165	16.68	27.50	30.44	— 2.94
Lowe Observatory	3 240	Ocean	5 000	165	14.75	24.35	25.18	— 0.82
Mill Creek	2 950	Ocean	10 000	145	16.83	24.40	24.50	— 0.10
Squirrel Inn	5 280	Ocean	6 000	215	17.96	38.60	37.32	+ 1.28

of the typical stations used in developing the diagram. Therefore, it can be assumed, inductively, that the use of this diagram would give reasonable values for the rainfall across these mountain ranges of varying crest height. Several reservoirs for power and irrigation have been constructed on top of these mountains, and in the determination of the feasibility of others projected, this diagram has been of considerable value as an index of the probable water supply from drainage areas with these variable characteristics as to precipitation conditions.

KENNETH ALLEN,* M. AM. SOC. C. E. (by letter).†—The author's comprehensive survey of the whole question of rainfall and run-off is a valuable contribution to the already voluminous literature on the subject. Relative to sewer design, to which this discussion is confined, a lamentable lack of uniformity in practice still exists, particularly with regard to run-off, an indication that certain elements in the computations are interpreted differently or are accepted with hesitation in the absence of complete fundamental data.

For an intensity formula, the expression, $I = \frac{C}{t^{\frac{1}{2}}}$, is recommended, and there is much to be said in its favor because of its simplicity and its fair accordance with observed results; but the writer believes that where long-time gaugings are available, modifications of this formula will often show a closer agreement with the curve plotted from gaugings.

The long series of self-registered gaugings at Central Park, New York City, covering 51 years, give an admirable opportunity for arriving at the probable frequency of different intensities, and the writer has attempted this by a method which is described in *Engineering News-Record* of April 7th, 1921.‡ Approximate formulas for the resulting curves shown in Fig. 11 are as follows:§

5-year curve........ $I = \frac{25}{(t + 4)^{0.67}}$

10- " " .... $I = \frac{43}{(t + 8)^{0.75}}$

15- " " .... $I = \frac{47}{(t + 8)^{0.75}}$

25- " " .... $I = \frac{78}{(t + 11)^{0.84}}$

50- " " .... $I = \frac{186}{(t + 17)}$

100- " " .... $I = \frac{354}{(t + 24)^{1.1}}$

Absolute maximum curve.. $I = \frac{74\ 000}{(t + 80)^2}$

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* New York City.

† Received by the Secretary, November 16th, 1921.

‡ Also referred to in *Proceedings*, Am. Soc. C. E., September, 1921, p. 232.

§ Suggested by Prof. Edward S. Allen, State College, Ames, Iowa, who sends the following more precise formulas for the 10 and 15-year curves:

$$\text{10-Year..}\ I = 27\,t^{-\frac{2}{3}}\ 20\,e^{-\frac{2}{9}t}\ t^{-\frac{4}{9}}$$

$$\text{15-Year..}\ I = 30\,t^{-\frac{2}{3}} - 32\,e^{-0.1t}\ t^{-1.1}$$

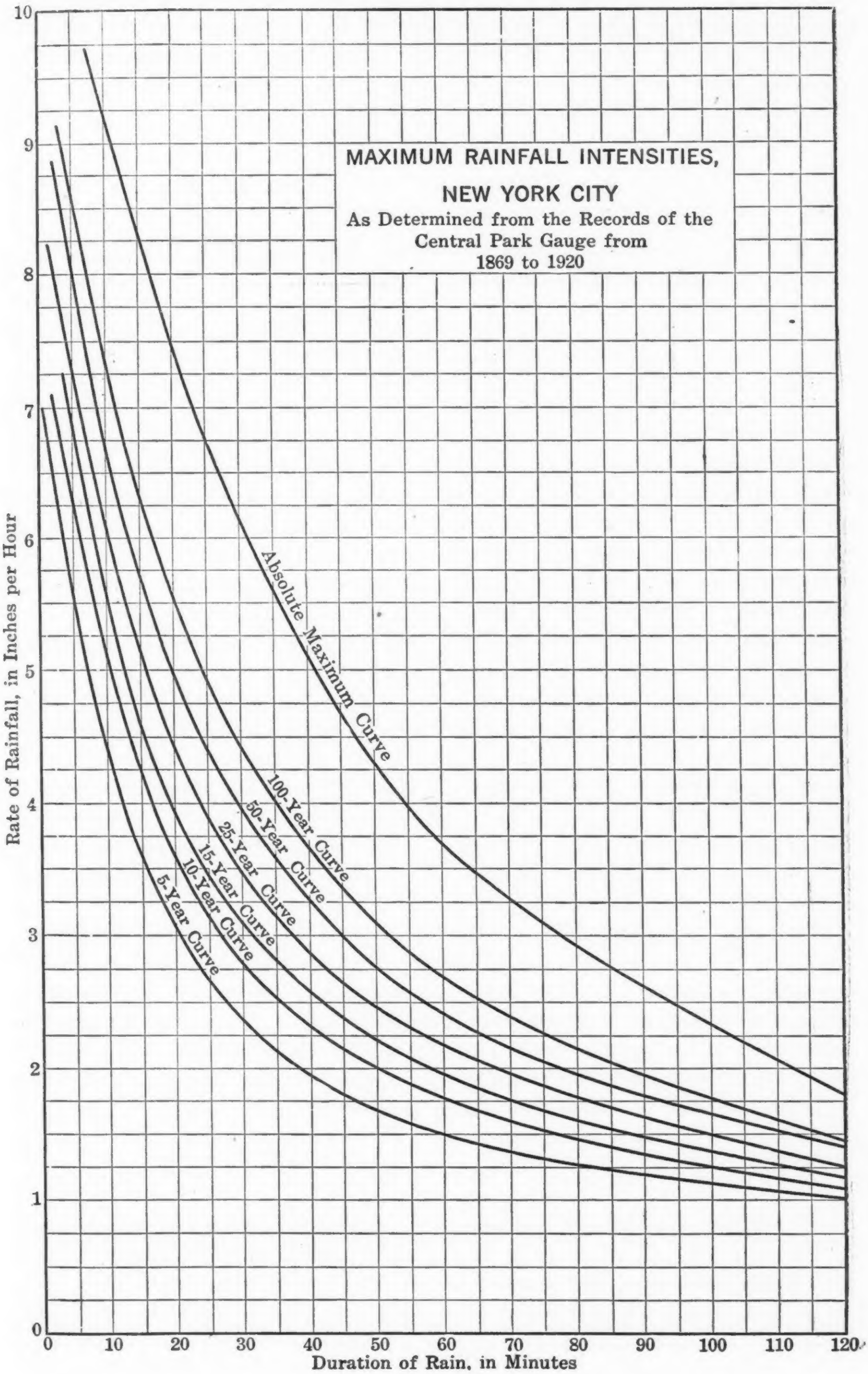
MAXIMUM RAINFALL INTENSITIES,
NEW YORK CITY
As Determined from the Records of the
Central Park Gauge from
1869 to 1920
Absolute Maximum Curve
100-Year Curve
50-Year Curve
25-Year Curve
15-Year Curve
10-Year Curve
5-Year Curve
Rate of Rainfall, in Inches per Hour
Duration of Rain, in Minutes
0
1
2
3
4
5
6
7
8
9
10
0
10
20
30
40
50
60
70
80
90
100
110
120

FIG. 11.

The author's Table 1 gives a comparison between his proposed "parabolic formula" and the "New York formula".* Table 14 is a comparison of these formulas with those determined by the foregoing formula for 10- and 15-year frequencies, and is confined for the most part to values of t used in sewer computations.

TABLE 14.—RAIN INTENSITY, IN INCHES PER HOUR.

t.	Brooklyn formula.	Parabolic formula.	10-year New York formula.	15-year New York formula.
5	6.0	6.5	6.28	6.86
10	5.0	4.6	4.92	5.38
20	3.75	3.24	3.53	3.86
30	3.00	2.65	2.81	3.07
60	1.88	1.88	1.82	1.98
120	1.07	1.32	1.13	1.24
720	0.20	0.54	0.31	0.34

The writer believes that the coefficients for run-off in general use should not receive the confidence which they are accorded, and has been unable to discover many recorded experiments for determining the percentage of run-off from areas of a single character. Although there is a general consensus of opinion among engineers as to run-off factors suitable for adoption, it is a question whether these have not been adopted from precedent rather than from knowledge. Mr. Grunsky has well said that "the effect of [this] perviousness must be made by the engineer, on the basis of experience".

Of the various elements entering into the determination of R from which erroneous results may be derived, the time of concentration and the gauging of the flow in the sewer are all important. Reliable information regarding the velocity of flow over different surfaces and along gutters is lamentably lacking, so that in practice the inlet time assumed is largely a matter of conjecture. Under urban conditions the error due to this is not often serious but, unless great care is taken, there are possibilities of serious errors in the computations of flow in the sewer, as any competent engineer knows.

Weirs are costly and difficult to introduce and the collection of débris may invalidate the results unless they are constantly under supervision; whereas dependence on computation by a formula may lead to serious error by varying hydraulic conditions, the selection of an improper friction coefficient, the assumption of the grades shown on construction plans instead of those which exist, and the complications introduced by sludge deposits, melting snow, bends, junctions, volume of domestic sewage, surface slope, the hydraulic effect of conditions above and below the section under consideration, etc. The fact that

* The author's use of the term, "New York Formula", for $I = \frac{150}{t + 20}$ is not strictly correct. Heretofore, each of the five Boroughs of the city has used its individual formula, and the foregoing formula adopted by the Borough of Brooklyn many years ago, is used by that Borough only. After studies resulting in a report of the Committee on Rainfall and Run-off of the Municipal Engineers of the City of New York and published on its *Proceedings*, for 1913, the Borough of Manhattan adopted the 10-year formula there proposed, namely, $I = \frac{150}{t + 16}$; but, in its application, t is taken to exclude the 4-min. inlet time which is added to the 16 min., giving it the form of the Brooklyn formula, but with a different value for t. In this manner the two Boroughs were inadvertably mentioned by the writer as using the same formula in his discussion of the paper by L. Standish Hall, Assoc. M. Am. Soc. C. E., in *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1919–20), p. 191.

more than 100% run-off is frequently shown in tabulated results is an indication that errors of the nature mentioned have been introduced.

The writer has recently had occasion to study the proper run-off factor for use under completely developed urban conditions and has made inquiry into the practise of various municipalities. Among those questioned there was quite a general agreement with percentages ranging roughly from 75 to 90 per cent.

Attention has been called recently to a number of densely built-up districts in a large city where the capacities of the main trunk sewers are only from 33 to 44% of those required for a 100% run-off and where no complaints of flooding have ever been received. This, of course, does not mean that these sewers were never surcharged, but the question at once arises: Why provide for 80% run-off if 40% answers every purpose?

The writer does not wish to appear as an advocate of the lower coefficients, but mentions this as an argument for more complete and reliable investigations than are now recorded and in such detail as to be entirely convincing.

The economical importance in the use of a proper run-off factor is obvious. Much of the cost of the sewer depends on whether a factor of 40, 60, or 80% is chosen and in a large district an expenditure of several hundred thousand dollars may depend on the selection. Any coefficient based on population density, although applicable in a general way for densities up to 100 per acre, as shown in Table 5, would be impracticable for such developments as obtain in the four cases previously cited, some of which house very large day, in addition to resident, populations.

To summarize, it is believed that although many experiments have been made to determine run-off, the published results usually fail to show that various possible causes of error have been eliminated and that there is need of more definite and reliable data regarding the proper run-off factors for areas of different types. Gaugings are costly and must be maintained for long periods and planned with great care. For these reasons, few municipalities undertake them, and it is submitted whether, in view of its importance and general nature, it would not be an appropriate investigation to be taken up by the Federal Government.

L. STANDISH HALL,* ASSOC. M. AM. SOC. C. E. (by letter).†—The author has presented many valuable data and formulas connected with rainfall and run-off studies, but only that part of the paper which deals with the relation between annual rainfall and run-off will be discussed.

The writer has been especially interested in the curve in Fig. 4, which shows the frequency of wet and dry years based on the combined records of rainfall at San Francisco and Sacramento, Cal. In presenting data of this nature, their value can be greatly increased if the coefficient of variation of the series is given, since this is an aid in comparing the results with similar curves that may be deduced from other data. In a recent paper the writer presented a series of curves showing the probable variation in yearly run-off

* Oakland, Cal.

† Received by the Secretary, November 23d, 1921.

covering a wide range of values of the coefficient of variation.* In order to compare the rainfall frequency given by Mr. Grunsky with the writer's run-off frequency curves previously mentioned, the coefficient of variation (*C. V.*) of Mr. Grunsky's series has been computed and found to be 0.35. Also, in plotting data of this kind, it is better to consider the percentage of time represented, rather than the number of years in the series, which will probably vary with each record to be studied. Plotting records on the basis of the percentage of time covered, therefore, is also an aid in reducing all records to a comparable basis.

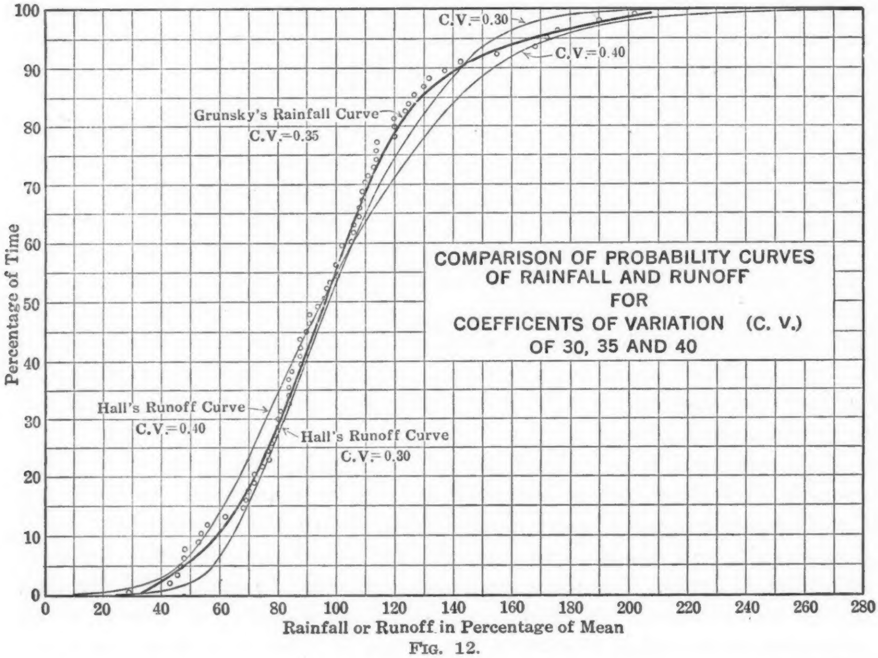

FIG. 12.

Mr. Grunsky's curve has been replotted in the manner suggested, and the writer's curves for values of *C. V.* of 0.30 and 0.40 are also shown in Fig. 12. The agreement between these curves is close, the greatest deviation being for values between 100% and 140% of the mean. It is possible that rainfall data can be represented best by a type of skew curve different from that which applies to run-off data of approximately the same coefficient of variation. However, it is unfortunate that more data were not utilized in constructing the rainfall frequency curve, since with the many long records of rainfall available in California, a series containing 500 terms or more could have been easily constructed, which would have indicated more definitely the trend of such a curve.

The accuracy of the long-time means determined from the San Francisco and Sacramento records, moreover, is open to question. These records represent

* See Fig. 13, "The Probable Variations in Yearly Run-Off as Determined from a Study of California Streams", *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 244.

a composite of records taken at different locations and with gauges under varying degrees of exposure. The greatest factor contributing to the error of the mean is that in these cities (and more especially in San Francisco), the gauges have been moved from their original locations near the ground to the tops of high buildings. Without going into the details which have already been given in previous papers,* it has been shown in the case of San Francisco, by comparing the official record with that kept by a private observer, that after the Weather Bureau had been moved to the top of the eleven-story Mills Building, the official record showed materially less catch than the private record, whereas prior to that time the two records had been in close agreement. When this subject was discussed previously, various arguments were advanced, defending the official record, but, at the present time, even the Weather Bureau officials recognize that the observations taken at the tops of high buildings do not reflect accurately the conditions near the ground; and it is now the policy of this Department to move their city stations, as fast as funds are available, to some open space where the instruments can be placed at or near the ground.

While on the subject of the effect of the exposure of rain gauges on the catch registered, attention may be called to an investigation by Messrs. William G. Reed and Howard M. Loy.† This study was originally undertaken for the purpose of making an intensive investigation of the relation between rainfall and run-off on a small drainage area. Strawberry Creek, a small intermittent stream which has a drainage area of about 600 acres, and which flows through the campus of the University of California at Berkeley, Cal., was selected for this purpose. The work was begun in 1913 when five gauges in addition to two permanent gauges on the campus were placed in different parts of the drainage area and read regularly. At the same time the flow of the creek was measured by a weir. The results obtained from the five rain gauges were so unsatisfactory that, in 1914, thirteen gauges were placed on the drainage area, in addition to the two on the campus. In Table 15 is shown the rainfall recorded in the various gauges between May 1st, 1914, and April 30th, 1915, together with the elevation of each gauge and the ratio of the catch of each gauge to the average. The term, expanded total rainfall, which is used, refers to the total recorded rainfall plus certain corrections to fill in gaps in the records at some of the gauges. The variation in the ratio of the catch of the various gauges for individual storms was often much greater than the average given in Table 15, and the records show that the lowest catches are in the gauges most exposed to the wind. It was stated as doubtful whether there was a single good rain-gauge exposure in the area under consideration, and it was believed that all the gauges registered less than the true rainfall. As a conclusion to this study, which was abandoned shortly afterward, Mr. Reed states:

* *Transactions*, Am. Soc. C. E., Vol. LXI (1908), pp. 526-529; and Vol. LXXXI (1917), p. 183.

† "The Water Resources of Strawberry Creek, Berkeley, Cal.", *Monthly Weather Review*, Vol. 43, pp. 35–39; and "Note of Effects of Raingauge Exposure", *Monthly Weather Review*, Vol. 43, pp. 318–322.

"From a strictly meteorological point of view the most important result so far seems to be the difficulty if not the impossibility of determining the precipitation on a water-shed by means of ordinary rain gauges."

TABLE 15.—RAINFALL OF STRAWBERRY CANYON FROM MAY 1ST, 1914, TO APRIL 30TH, 1915, WITH RATIOS OF INDIVIDUAL GAUGES TO THE AVERAGE.

Gauge number.	Elevation, in feet.	Expanded total rainfall, in inches.	Ratio to average.
Students' Obser	325	26.91	1.00
Civil Engineering Building	410	24.49	0.91
1.	520	28.79	1.07
2.	730	28.79	1.07
3.	880	26.91	1.00
4.	1 225	26.10	0.97
5.	1 270	22.60	0.84
5.	1 270	21.53	0.80
6.	1 250	27.18	1.01
7.	1 190	25.30	0.94
8.	1 180	28.26	1.05
9.	915	29.06	1.08
10.	1 210	29.06	1.08
11.	1 315	32.03	1.19
12.	1 655	29.06	1.08

It would appear, therefore, that great doubt exists as to the accuracy of long-time means based on composite records. Hence, it would seem that there is nothing to be gained by expanding rainfall records to agree with the long-time means at San Francisco and Sacramento.

It may be of interest to discuss in some detail the results of a study of the relation between rainfall and run-off on Putah Creek, as this stream shows some of the typical difficulties encountered in studies of this nature.

The records of the gaugings of the flow of Putah Creek at Winters, Cal., are available for a period of fifteen years from 1905-06 to 1919-20. The record for a period of nine years has been presented by the writer.* The run-off of the remaining six years need not be shown, except to state that, in 1919-20, the run-off was slightly less than in 1911-12, and that the mean run-off for the fifteen years is 420 000 acre-ft. In order to expand this record over the period covered by the rainfall records, the precipitation by seasonal years for eighteen stations in the neighborhood of the drainage area were collected. After gathering these data, it was found that only seven stations out of the eighteen had complete records for the fifteen years that run-off records were available. Gaps in the records were filled by estimating the missing years from the rainfall at neighboring stations.

Table 16 gives the names of the various rainfall stations, together with the elevation and the mean annual rainfall for the 15-year period. Only one of the eighteen stations, that at Helen Mine on the north side of Mount St. Helena, is within the catchment area, and at this station the mean rainfall is 82.3 in., or more than twice the mean rainfall at any of the other stations. Helen Mine is the only station on a mountain slope, all the other stations being in the bottoms of valleys. Without this station an erroneous idea of the true mean rainfall in the Putah Creek basin would have resulted. From the scarcity and wide distribution of the rainfall stations it is almost impossible

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), Stream 11, Table 1, p. 194.

to estimate the mean annual rainfall on this drainage area with any degree of accuracy, although it is probable that it is between 30 and 40 in. In cases of this kind, better results can be obtained by disregarding the actual rainfall and determining the relation between the percentage of rainfall and the run-off.

TABLE 16.

Station.	Elevation, in feet.	Mean annual rainfall 15-year period, in inches.	Station.	Elevation, in feet.	Mean annual rainfall 15-year period, in inches.
Suisun	15	19.80	Peachland	190	39.59
Napa	50	23.12	St. Helena	255	35.32
Davis	51	17.24	Cloverdale	340	38.89
Healdsburg	52	38.66	Guinda	350	20.37
Woodland	63	16.24	Calistoga	363	36.98
Dunnigan	65	19.07	Ukiah	620	36.75
Sacramento	71	15.95	Upper Lake	1 350	27.16
Vacaville	175	24.73	No. Lakeport	1 450	28.12
Santa Rosa	181	28.42	Helen Mine	2 750	82.29

In Fig. 13 is shown the relation between mean annual rainfall and mean annual run-off for Putah Creek, based on the fifteen-year period covered by the gaugings. The relation between the annual rainfall and run-off has not

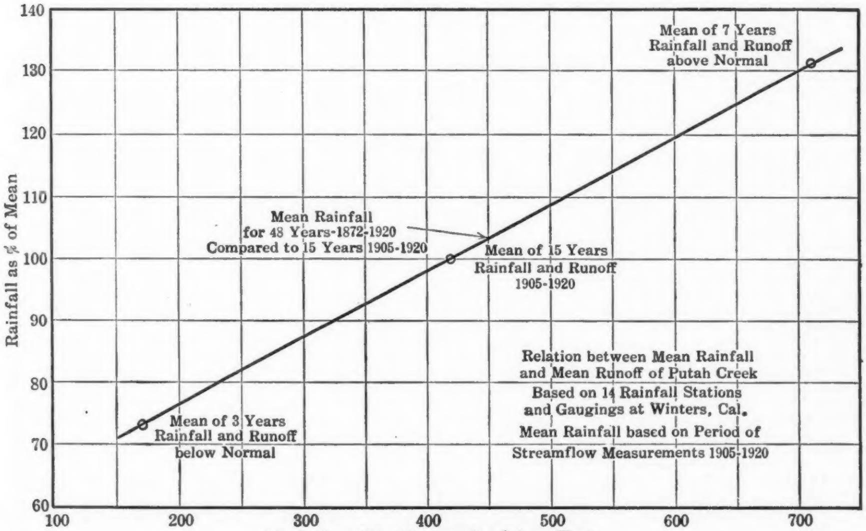

FIG. 13.

been shown in the diagram, because the only use that is made of the rainfall run-off relation in this discussion is to determine the relation between the observed and the probable run-off for the period covered by the rainfall records. By using the data in the manner illustrated in Fig. 13, it has been found that in practically all cases the relation between the percentage of mean annual rainfall and the run-off can be represented by a straight line.

The percentage of rainfall for each year of the records at each station was calculated on the basis of the 15-year means, and these percentages were then

averaged to obtain the average percentage of rainfall for each year. For the 48 years, 1872-73 to 1919-20, there were always at least three stations available for computing the average, but for the 23 years, from 1849-50 to 1871-72, only the record at Sacramento was available. Using the records in this manner, it appeared that for the 48-year period the rainfall was 103.5% of that in the 15-year period, and that for the 71 years it was 109.0% of that in the 15-year period.

From a closer study of the records, it would appear that the conditions which have affected the catch registered by the San Francisco gauge in recent years, that is, moving the gauge to the top of a high building, has affected the Sacramento record in a similar manner. In support of this statement, Table 17 is offered, showing the ratio between the mean rainfall for the fifteen years, 1905-06 to 1919-20, and the mean for the entire record at each station where the records were of sufficient length to warrant making the comparison. The record for Sacramento is shown for two periods, 48 years and 71 years.

It will be noted from Table 17, that the ratio of the rainfall for the entire record to that of the last fifteen years is disproportionately greater in the case of Sacramento than in that of any other station, if either the 48 or the 71-year period is considered. In view of these results, it would appear that nothing but erroneous results could be obtained by expanding short records to agree with the long-time means at San Francisco and Sacramento.

TABLE 17.—SUMMARY OF RAINFALL RECORDS NEAR PUTAH CREEK DRAINAGE AREA.

Station.	Mean, 15 years, 1905-20, in inches.	Years in record.	Ratio of mean, entire record, to mean, 15 years.
Sacramento	15.95	48	115.9
Sacramento	15.95	71	117.6
Davis	17.24	48	98.8
Vacaville	24.73	40	106.7
Suisun City	19.80	46	101.9
Napa	23.12	43	103.5
Dunnigan	19.07	43	103.5
Woodland	16.24	47	108.1
Upper Lake	27.16	34	101.1
Calistoga	36.98	47	99.3
Ukiah	36.75	43	99 5
Healdsburg	38.66	43	107.4
Santa Rosa	28.42	32	106.3

It was decided, therefore, in the case of the Putah Creek study, that the results obtained from the 48-year period were much more dependable than those for the 71-year period. Using the result for this period and Fig. 13, it would appear that the mean annual run-off for the 48 years should be 450 000 acre-ft. rather than the 420 000 acre-ft., as determined from the measured flow.

In Table 4, Mr. Grunsky has presented the probable range in rainfall and run-off to be expected, in a period of 100 years, from a water-shed in the vicinity of San Francisco having a mean annual rainfall of 30 in. Similar results may be obtained for the run-off—which is really the result finally desired—by using the writer's frequency curves for run-off.* For streams in the vicinity of San Francisco, the curve for the coefficient of variation of 0.70

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 244.

is the proper one to use. In order to support this statement, statistics on some of these streams will be set forth in Table 18, and it will be noted that although some differences occur in the run-off, the coefficient of variation is very consistent and varies but little from 0.70.

TABLE 18.—SUMMARY OF RUN-OFF RECORDS IN THE VICINITY OF SAN FRANCISCO, CAL.

Stream.	Place.	Years in record.	Area, in square miles.	Mean run-off, in inches.	*C. V.*
San Leandro	Oakland	34	42	9.0	0.72
Crystal Springs	San Mateo	30	36	11.1	0.69
Alameda	Sunol	29	620	4.8	0.72
Coyote	Madrone	13	193	7.5	0.80
Putah	Winters	14	805	10.5	0.67
Cache	Yolo	16	1320	7.8	0.74

Applying this method to Putah Creek, Table 19 shows an ideal 100-year record of annual run-offs for this stream, arranged in order of magnitude, and based on a mean annual run-off of 450 000 acre-ft., and a coefficient of variation of 0.70.

TABLE 19.—IDEAL 100-YEAR RECORD OF THE ANNUAL RUN-OFF OF PUTAH CREEK.

Order of magnitude.	Percentage of mean.	Run-off, in acre feet.	Order of magnitude.	Percentage of mean.	Run-off, in acre feet.	Order of magnitude.	Percentage of mean.	Run-off in acre feet.
1	3	13 000	35	58	261 000	68	120	540 000
2	8	36 000	36	60	270 000	69	123	553 000
3	11	49 000	37	61	275 000	70	125	563 000
4	14	63 000	38	62	279 000	71	128	576 000
5	16	72 000	39	63	283 000	72	131	589 000
6	18	81 000	40	65	293 000	73	134	603 000
7	20	90 000	41	67	301 000	74	137	616 000
8	22	99 000	42	68	306 000	75	140	630 000
9	24	108 000	43	69	311 000	76	143	643 000
10	25	113 000	44	71	319 000	77	146	656 000
11	27	121 000	45	72	324 000	78	149	671 000
12	29	131 000	46	74	333 000	79	153	687 000
13	30	135 000	47	75	337 000	80	157	707 000
14	32	144 000	48	77	347 000	81	161	725 000
15	33	149 000	49	79	355 000	82	164	738 000
16	34	153 000	50	80	360 000	83	168	756 000
17	36	162 000	51	82	369 000	84	172	774 000
18	37	167 000	52	83	373 000	85	177	796 000
19	38	171 000	53	85	383 000	86	181	814 000
20	40	180 000	54	87	391 000	87	186	836 000
21	41	185 000	55	89	401 000	88	190	855 000
22	42	189 000	56	91	409 000	89	195	877 000
23	44	198 000	57	93	419 000	90	200	900 000
24	45	203 000	58	95	427 000	91	206	926 000
25	46	207 000	59	97	437 000	92	210	945 000
26	47	211 000	60	100	450 000	93	217	977 000
27	48	216 000	61	102	459 000	94	224	1 008 000
28	50	225 000	62	105	473 000	95	231	1 039 000
29	51	229 000	63	107	481 000	96	240	1 080 000
30	52	234 000	64	110	495 000	97	250	1 125 000
31	54	243 000	65	112	504 000	98	262	1 168 000
32	55	247 000	66	115	517 000	99	285	1 283 000
33	56	252 000	67	118	531 000	100	338	1 521 000
34	57	257 000						
Mean							100	450 000

The percentages given in Table 19 will apply equally well to any stream having a coefficient of variation of 0.70, and the yearly run-off can be obtained by multiplying these percentages by the mean annual flow. The method of obtaining these percentages has been explained in the writer's paper previously mentioned.* The minimum run-off to be expected once in 100 years on Putah Creek at Winters, or 13 000 acre-ft., is equivalent to a depth of 0.30 in. on the drainage area, while the maximum run-off, or 1 521 000 acre-ft., is equivalent to a depth of 35.5 in. on the drainage area. The figures given in Table 19 may be used in constructing an artificial record of any desired length, which will be comparable to an actual record of similar length obtained under natural conditions.* Such a record is interesting and instructive, as from it the probable frequency of the re-occurrence of any series of dry years can be foretold with reasonable accuracy.

Some space has been devoted to a discussion of the desirability of having the Weather Bureau publish rainfall totals by climatic or seasonal years rather than by calendar years. Although the annual summaries published by the Weather Bureau give the precipitation by calendar years only, bulletins of climatological data are issued at intervals of a few years, in which the entire record of rainfall at each station to the the date of publication are given both by seasonal and by calendar years. In order to bring the records to date, it is generally only necessary to secure the seasonal totals over a short period of years.

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* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), pp. 250–251.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE RELATION BETWEEN DEFLECTIONS AND STRESSES IN ARCH DAMS

Discussion*

By L. J. Mensch, M. Am. Soc. C. E.

L. J. Mensch,† M. Am. Soc. C. E. (by letter).‡—During a recent visit to the Pacific Coast, the writer was greatly impressed by the great importance of dams to the prosperity of the western part of the United States. For irrigation, power development, and flood control, dams of enormous heights are proposed, among which may be mentioned the Pacoima Creek Dam, in the San Fernando Valley, north of Los Angeles, 375 ft. high, the St. Gabriel Dam, 15 miles west of Los Angeles, 420 ft. high, and the gigantic Boulder Canyon Dam of the Colorado River, with a height above bed-rock of 730 ft., about 200 ft. long at the base, and 1 000 ft. long at the crest. The sparsely settled West can hardly afford gravity dams of these great dimensions.

Water and power have always been absolute necessities for the development of the West and capital has been scarce, therefore, new types of dams came in vogue, which would have been out of the question in more settled countries. Such dams are the rock-fill dam, the hydraulic-fill dam, the multiple-arch dam, and the single-arch dam (notably represented by the Bear Valley Dam built in 1884 and the Upper Otay Dam built in 1900, both unsurpassed for economy by any other dam or for safety by few gravity dams). When it is considered that such a simple problem as the design of a gravity dam required more than 100 years of experience and many important failures before it was partly solved so that engineers agreed fairly well on the main features of the design, the fact that the design of new types of dams is not free from many imperfections need not cause much surprise. These dams are usually designed

* This discussion (of the paper by F. A. Noetzli, Assoc. M. Am. Soc. C. E., published in October, 1921, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chicago, Ill.

‡Received by the Secretary, November 12th, 1921.

by hydraulic engineers who have their own particular problems to develop and who, as a rule, have not the time, inclination, nor experience and perception, not to speak of the important knowledge of the literature on the subject, to delve into the intricacies of strange and difficult structural problems.

Engineers trained in structural design look, with wonderment, and also with suspicion, at the apparently crude principles by which most of these dams have been designed, and Mr. Noetzli's paper may be considered as a plea against dam design which is not based on strictly structural lines and more especially against the practice of designing single-arch dams on the principle of a thin cylinder. Scientific principles of the co-related problem of the design of cylindrical tank walls where they join the bottom were published many years ago. Grashof treats of it in Paragraphs 203 to 210 of his book.* A graphical solution is given by Dr. von Emperger† for a tank 130 ft. in diameter and 28 ft. high. A very good and simple approximate solution is given in *Beton und Eisen,* 1907, page 257. Professor Reissner also gave a more rigorous treatment of the subject in *Beton und Eisen,* 1908, page 226, which is nearly identical with the solution presented to the Society twelve years later by B. A. Smith, M. Am. Soc. C. E.‡ Since then at least a dozen articles on the same subject have appeared in *Beton und Eisen.* An approximate solution was published in *Engineering and Contracting* in 1912 for a tank built by the writer in Penetanguishine, Ont., Canada.

That large tanks, built of masonry or even of steel, which are not designed for the restraint of the walls at the bottom, are not satisfactory may be shown by the collapse of a large molasses tank in Boston, Mass., a few years ago and the trouble caused by cracks in concrete tanks where the walls join the bottom. A serious case came to the writer's attention a few years ago. A certain Middle Western town contracted for two reinforced concrete tanks 150 ft. in diameter and 24 ft. high. The wall was 24 in. thick at the bottom and was keyed into a recess of the foundation. After the first tank was finished, water was admitted, but the wall sheared off the key and, besides, cracked the floor of the tank near the wall. On the writer's advice a number of circumferential bars near the bottom were omitted in the construction of the second tank and cut up into shorter bars which served as reinforcement of the joint between the wall and the bottom to take up the cantilever moment at the base. The second tank, although built by the same contractor, was tight from the first filling.

The writer attempted to use the same theory for the design of single-arch dams, and was surprised to find that according to this theory nearly all existing arch dams ought to show serious defects. Although a few such dams in Australia have shown some vertical and horizontal cracks, they evidently are not serious, and few cracks except those which may be ascribed to temperature action have been reported on dams built in the United States. No considerable leakage at the bottom, like that so often observed in reinforced concrete tanks where the walls are not properly tied to the bottom, have been reported for

* "Theorie der Elasticität und Festigkeit", Berlin, 1878.

† "Handbuch für Eisenbetonbau", Vol. 3 (1907).

‡ *Transactions,* Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 2027.

any arch dam; on the contrary, many arch dams are absolutely dry on the down-stream side.

The author's praiseworthy efforts to solve the problem failed, in the writer's opinion, because he tried to solve only that part which has been understood fairly well by structural engineers and which, as has been stated, does not explain the well-known safety of arch dams. He tried to explain the arch action by the theory of the thin parabolic arch of small rise, although every textbook containing this theory warns the student not to use it in case of thick arches or arches of considerable rise. Serious discrepancies having been called to his attention in the discussion of his first paper,* he tried to improve his theory by using the formulas of a thick parabolic arch with small rise, which is not sufficiently accurate for the lower arches of a dam. In his excellent paper on thick circular arches† Professor Cain has shown that Mr. Noetzli's formula for deflection is not reliable for flat arches, and the writer can affirm that the curve in Fig. 4 for the 180° arches gives values at least 50% too great and the other curves are in error in proportion. In the upper part of a dam, the secondary stresses are comparatively small and, for that part, the author's formulas give workable results.

How untenable the author's approximate theory is will be illustrated by the example he chose, the Salmon Creek Dam, designed by L. R. Jorgensen, M. Am. Soc. C. E. The author has shown that the cantilever broke when the water rose to the top of the dam; he also has shown that according to his views the deflection of the ring 28 ft. above the base, was considerably larger than it would be according to the cylinder theory; therefore, nothing less than the water pressure should act at that elevation. Even if by a more judicious selection of the modulus of elasticity, a larger deflection might be calculated by the cylinder theory, the actual deflection is so large that no reputable engineer will dare to be satisfied unless the arch ring is strong enough to sustain the entire water pressure. What, according to the author, happens in such a case? The negative thrust or pull (the author seems to speak of a positive thrust), due to the shortening of the arch, according to his Equation (1*a*), is:

$$H = 0.75\, f'_c + \frac{t^3}{h^2}$$

Mr. Jorgensen gives‡:

$$f_c' = 300 \text{ lb. per sq. in.}$$
$$t = 36 \text{ ft. and}$$
$$h = 28 \text{ ft., approximately.}$$

Therefore,

$$H = 0.75 \times 144 \times 300 \times \frac{36^3}{28^2} = 2\,000\,000 \text{ lb. per lin. ft.}$$

* "Gravity and Arch Action in Curved Dams", *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

† *Proceedings*, Am. Soc. C. E., October, 1921, p. 285.

‡ *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), Plate XIII, p. 709.

This pull produces at the abutment a moment of

$$2\,000\,000 \times \left(\frac{2}{3} \times 28 \text{ ft.}\right) = 37.3 \times 10^6 \text{ ft-lb.}$$

According to the author, the extreme fiber stresses equal

$$\frac{37.3 \times 10^6}{\frac{36^2}{6}} = \pm\ 172\,000 \text{ lb. per sq. ft.}$$

The pull of 2 000 000 lb. produces according to Mr. Noetzli's reasoning a tension of

$$\frac{2\,000\,000}{36} = \ldots\ldots\ldots -55\,500 \text{ lb. per sq. ft.}$$

or a maximum tension of 227 500 lb. and a compression of 116 500 lb. per sq. ft. To these stresses are to be added the compressive stresses from the pure arch action of $300 \times 144 = 43\,200$ lb. per sq. ft., giving a maximum tension of 184 300 lb. and a maximum compression of 159 700 lb. per sq. ft. Evidently, according to the author's approximate theory, the arch ring cannot carry the load; still, the dam has not shown any sign of failure.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE CIRCULAR ARCH UNDER NORMAL LOADS

Discussion*

BY L. J. MENSCH, M. AM. SOC. C. E.

L. J. MENSCH,† M. AM. SOC. C. E. (by letter).‡—This paper is a valuable addition to the literature on arches, and will dispel speedily and effectively many wrong ideas of designers of past and prospective arch dams.

In his discussion§ of the paper by H. Hawgood, M. Am. Soc. C. E., entitled, "Huacal Dam, Sonora, Mexico", the writer has treated the thick arch in a similar manner. He designated the expression, $pr - P_0$, which is really a negative thrust or pull due to the shortening of the arch or a fall of temperature, by T, and believes to have made the action of the shortening of the arch appear more clearly than by compounding the whole action by the method of Castigliano. Since that time the writer has found that it is not permissible to neglect the effect of shear for the lower arches of a dam and has been agreeably surprised that the formulas for thrust and deflection have not become more complicated and, in some cases, are even simpler than those where shear is neglected. For example, the author's Equation (11) becomes, when the effect of shear is included;

$$D = \left(\frac{1}{4} \sin 2\,\phi_1 + \frac{\phi_1}{2}\right) 2\,\phi_1 + \frac{k^2}{r^2}\left(2\,\phi_1 - \frac{1}{2} \sin 2\,\phi_1\right) 2\,\phi_1 - 2 \sin^2 \phi_1$$

Any process used to obtain formulas for thrust, moment, and deflections, is always complicated on account of the great length of the equations, the many fractions, plus and minus signs, indices, decimals, and the most skillful mathematician is led into many errors. The writer never considers a new problem solved, until the same result has been obtained by different processes at an interval of a year or more, or when the result is obtained by three persons working independently.

* This discussion (of the paper by William Cain, M. Am. Soc. C. E., published in October, 1921, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Chicago, Ill.

‡ Received by the Secretary, November 12th, 1921.

§ *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 610.

The work of preparing a paper like that of the author is very great, and only those who have tried similar original investigations have any idea of the energy expended on it. It is not surprising, therefore, that such a paper should contain errors. The writer questions Equation (18) and some of the deflec-

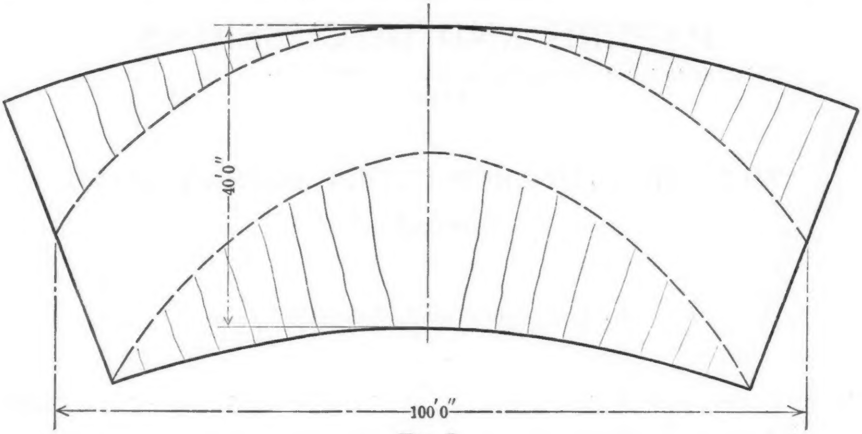

FIG. 5.

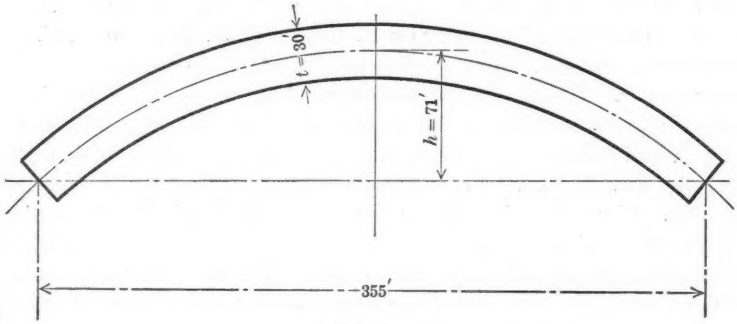

FIG. 6.

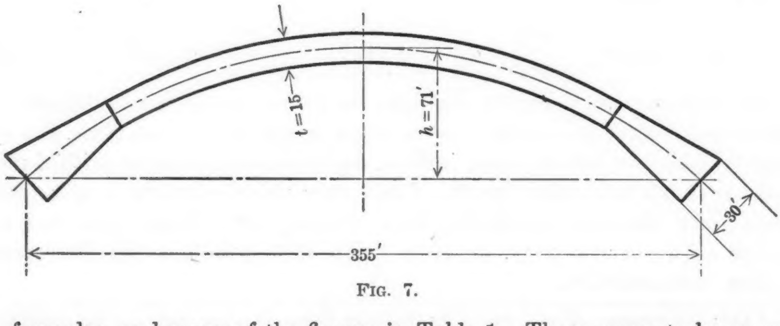

FIG. 7.

tion formulas, and many of the figures in Table 1. There seems to be an error in calculating P_0 for the arches 40 ft. thick, in the examples chosen by the author, because the values of $pr - P_0$ ought to be approximately one hundred times greater than for the arches, 4 ft. thick. Arch action, as commonly

understood, is small in flat thick arches, and a glance at a flat arch, as shown in Fig. 5, taken from an existing arch dam, will show that the effect of the horizontal components of the water pressure must be very small, and the simplest assumption is that the pressure is taken up by beam action. Can the beam carry this load? This question has not been considered by most writers on the subject. In this case the water pressure amounts to 10 000 lb. per sq. ft. The corresponding bending moment equals $10\,000 \times \frac{100^2}{24} = 4.17 \times 10^6$ ft-lb. The moment of resistance is $\frac{40^2}{6}$ and the extreme fiber stresses, in pounds per square foot are $\mp$ 15 000 at the crown and $\mp$ 3 000 at the abutments. It cannot be assumed that compression from the arch will reduce these tensile stresses very much, because the arch cannot deflect sufficiently to compress the fibers before they are cracked, or before cracks appear at the construction joints. The beam will show cracks, somewhat similar to those illustrated in Fig. 5. A sound part of the beam will remain in the shape of a new arch which can act as a real arch, and the size of this new arch may be determined by the theory of least work. It might be objected that such cracks if they really existed would have been reported on dams already built. Calculations, however, show that the sum of the width of all the cracks may be only $\frac{1}{7\,000}$ in., hence they would be considered to be hair-cracks only, if found at all. This action of the thick flat arch has been only vaguely understood and denominated as "wedge" action, although a wedge is not a recognized structural member.

It is clear that the new arch will show much higher stresses, than those calculated on the whole section by the ordinary cylinder theory. Professor Cain, also Mr. Noetzli, believes that the stresses can be found from deflection measurements of the arch. The writer thinks that this method would lead to erroneous views. The modulus of elasticity is a greatly varying quantity. Even in a common cylinder test, the moduli on different sides of the cylinder often vary more than 100% and the figures given by authors are only averages. The best information on this subject is found in the report of the Committee of the Austrian Engineering Society on tests on a number of masonry and reinforced concrete arches (made in 1892 at an expense of $20 000) and the scientific treatise by Mr. J. A. Spitzer on the reinforced concrete arches of these tests.* Arches were loaded with known weights, acting at known points, and the deflections and changes of angles were observed carefully at many points. From the deflection, the modulus of elasticity was calculated to be 2 000 000 lb. per sq. in., up to a load about 30% less than that producing the first visible crack. At the first crack, the modulus was about one-half the value mentioned, and for higher loads was still much less.

There is another uncertainty about the modulus of elasticity, due to the flow or plasticity of concrete, and many phenomena cannot be explained unless a low modulus is assumed. Such is the case with the effect of a change in temperature or the shrinkage of the concrete, which are slow actions, in which

* *Zeitschrift*, Osterreichischer Ingenieur und Architecten Verein, No. 20, 1896.

the time factor plays an important rôle. Recent tests on the shrinkage of concrete and on the effect of temperature lead to the conclusion that the modulus of elasticity lies between 500 000 to 1 000 000 lb. per sq. in. for these actions and, in some cases, drops to 100 000 lb. per sq. in. and less. The stresses cannot be found from the deflections in an arch with contraction joints or cracks (unless the arch is properly reinforced), because the distribution of stresses in those planes is widely different from that obtained by the straight-line theory, once tensile stresses are calculated in the extreme fibers.

In this connection the writer wishes again to call the attention of the Profession to the unscientific character of the plain masonry or concrete arch dam, as commonly built. Fig. 6 shows a horizontal section of an arch dam which has been constructed. A non-reinforced arch like this might have been designed by the old Roman engineers. A modern bridge designer would never use it, because the stresses from temperature and shrinkage action will be larger than those from the live loads, and such an arch will crack. The latest information available indicates that the greatest shortening due to the combination of a fall in temperature and shrinkage of concrete for an arch of that thickness is about $\frac{1}{1\,800}$ of the length. The pull or negative thrust caused by this shortening, when the modulus of elasticity is 1 000 000 lb. per sq. in., can be obtained for an arch of that particular relation of rise to span by the formula:

$$H = \frac{66\,000\,t}{\left(\frac{h}{t}\right)^2 + 1.26} \text{ lb. per lin. ft.}$$

when h equals the rise of the arch and t the thickness, in feet. This formula has been obtained in a similar manner, as shown in the writer's discussion of the Huacal Dam, previously referred to, except that the effect of shear was considered. By substituting $t = 30$ ft., $h = 71$ ft., $H = 9\,550 \times t = 286\,500$ lb. per lin. ft., and using Mr. Noetzli's approximation of $\frac{2}{3} \times t$ as the leverage of this pull on a section at the abutment, the moment there becomes:

$$286\,500 \times 47.33 = 13.55 \times 10^6 \text{ ft-lb.}$$

The moment of resistance of the section being $\frac{30^2}{6}$, the extreme fiber stresses at the abutment are $\pm$ 90 500 lb. per sq. ft. The component of the pull, tangent to the arch at the abutment, is about 220 000 lb. and the corresponding tension, $\frac{220\,000}{30} = 7\,333$ lb., or the extreme fiber stresses, — 97 833 lb. and + 83 167 lb., respectively. The arch will crack or expansion joints will open, which fact is amply confirmed by actual observation of that dam. The water pressure does not entirely relieve the arch of the tensile stresses. It produces only a compression of 47 000 lb. per sq. ft.; therefore, the arch must act as a reduced section, as previously described for the very thick arch, with much higher stresses than the 47 000 lb. assumed by the designer.

Assume, now, that the arch has a section similar to that shown in Fig. 7. On account of the enlargement at the abutment, the negative thrust will be comparatively larger than that for an arch of a uniform thickness, and approximately is given by:

$$H = \frac{100\,000\,t}{\left(\frac{h}{t}\right)^2 + 2}$$

or,

$$H = 61\,500 \text{ lb.}$$

The leverage of this force about a section at the abutment also becomes greater and will be assumed to be 50 ft.; therefore, the moment at the abutment is $61\,500 \times 50 = 3\,075\,000$ lb., and the extreme fiber stresses are $\dfrac{3\,075\,000}{\frac{30^2}{6}} = 20\,500$ lb. per sq. ft. The direct tensile stresses are approximately 1 440 lb., and the total stresses are $-$ 21 940 and $+$ 18 060 lb., respectively. The compressive stresses from the water pressure being 47 000 lb. at the abutment will more than counterbalance the tensile stresses, but will not prevent the opening of contraction joints when the dam is empty. This can be prevented only by properly reinforcing the arch and anchoring the reinforcing bars into the rock walls at the abutments. The writer hopes to have shown that with a saving of nearly 50% in masonry, lower stresses are produced in a reinforced concrete dam than in the plain concrete arch of to-day, and this without the fear of dangerous cracks.

That reinforced concrete arches are much more economical than plain concrete arches was unquestionably brought out by the Austrian tests, previously mentioned. A concrete arch of 76 ft. span, 15 ft. rise, 6.5 ft. wide, and 27.5 in. thick throughout, was tested to failure by the application of a total load of 185 000 lb., uniformly distributed on one side of the arch.

A reinforced concrete arch of the same span, rise, width, with a crown thickness of only 13¾ in., increasing to 25 in. at the abutment, failed at a much higher load, namely, 322 000 lb., distributed on the half span.

The arch dam of the future should not show larger cracks than modern arch bridges, and with richer concrete and proper reinforcement, there is no reason why stresses as high as those calculated for other reinforced concrete work should not be used. Thoughtful engineers may object to the use of reinforcement on account of the danger of the steel rusting by the action of intrusive water; there is no more danger that this will happen in dams than in reinforced concrete water tanks and reinforced concrete sewers, the general use of which is older than modern reinforced concrete buildings and bridges. Of course, richer concrete must be used, and the face of the dam should be water-proofed by any of the excellent methods which have proved successful in tunnels and high tanks. It is a fact, that most reinforced concrete tanks, in which the concrete acts in tension only, are comparatively much thinner than most concrete arch dams. Should similar structures in which the concrete acts primarily in compression be made thicker for the only reason that there may be some similarity with a gravity dam?

According to the writer's opinion every arch ring of a dam should be designed to carry the entire water pressure, including all secondary stresses, proper allowance should be made since the effect of shrinkage and temperature in thick sections is very much less than in thin sections, as the concrete in the interior has no chance to dry out or to become affected to any extent by a change in temperature, and cracks should be prevented by proper reinforcement.

In large arch dams, as a rule, it is uneconomical to assume the arch rings to be affected only by the water pressure. If the weight of the arch ring is also considered, it will be found that the most economical design is obtained by inclined arches, the inclination being given by the ratio of water pressure per square foot to total weight of the inclined ring per linear foot. The reinforced concrete arch dam should be considered as a monolith, affected by the entire water pressure, the weight of the structure, and the inclined reactions of the side-walls. The inclined rings will take up most directly the inclined reactions and will relieve the base of the dam from the superimposed load; therefore, the supposed favorable influence of Poisson's ratio cannot be considered when the reservoir is full and the high stresses from the cantilever action do not exist at the base in a reinforced concrete dam. The rings have different inclinations at different depths and become nearly horizontal at the base of the dam.

During a stay on the Pacific Coast, the writer made a design on these lines for a reinforced concrete arch dam 210 ft. high, and 400 ft. long at the river bed, and found that a section 35 ft. thick at the base had much smaller stresses than would be found in a plain concrete curved dam of twice the thickness.

Engineers may be afraid to use comparatively thin, reinforced concrete arches on account of the supposed column action in such slender arch rings. The writer knows of only one test of a concrete arch where failure was produced by an uniform load over the entire span. This test was made by the Blaubeuren cement mill, in Ehingen on the Danube, on a three-hinged arch of 66 ft. span, 5 ft. rise, $4\frac{3}{4}$ in. thick at the crown, $8\frac{3}{4}$ in. thick at the abutment, and $12\frac{1}{2}$ in. thick at the $\frac{1}{4}$-in. point. The structure was loaded for 7 years from 1896 to 1903, and failed after the last increase of load had been on it for 4 years. The maximum compressive stresses were calculated to be 840 000 lb. per sq. ft., although 12-in. cubes, cut from the structure afterward, failed at 954 000 lb. A reduction of compressive stress is found in tests of short columns, and the test of this slender arch does not indicate that column action is to be feared in arched dams.

The writer hopes that Professor Cain's paper will be considered carefully by all engineers interested in dam construction and will induce them to use more science and less speculation in the design of single and multiple-arch dams.

AMERICAN SOCIETY OF CIVIL ENGINEERS

INSTITUTED 1852

PAPERS AND DISCUSSIONS

THE FLOOD OF SEPTEMBER, 1921, AT SAN ANTONIO, TEXAS

Discussion*

By Charles W. Sherman, M. Am. Soc. C. E.

Charles W. Sherman,† M. Am. Soc. C. E.—The information contained in this paper is of especial interest to the speaker because of his intimate connection with the preparation of the Metcalf and Eddy report to which Mr. Bartlett has referred.‡

This report embodied the results of a special study of local conditions, including such information relating to previous floods in San Antonio River as could be obtained, and a detailed examination of the records of intense rains in Texas and of the flood discharge of streams throughout North America.

As Mr. Bartlett has stated, floods which did considerable damage along the San Antonio River, occurred twice in 1913, and one which just escaped doing material damage occurred in 1914. Reasonably satisfactory information regarding these floods was obtainable, but data on earlier floods were extremely fragmentary and unsatisfactory. The dates of their occurrence and some approximation as to the height of the flood water at a few points were all that could be obtained, and much of this information did not become available until the investigation was nearly completed. Therefore, the estimation of the magnitude and frequency of serious floods had to be made on very incomplete information, and estimates of possible floods based on rainfall records proved of material significance in reaching the final conclusion.

The report recommended flood-control works for the San Antonio River, including an improved river channel with a capacity of 12 000 cu. ft. per sec. and a detention basin above the city sufficient to retain an additional 10 000 cu. ft. per sec., so that the complete works would care for a flood of approxi-

* This discussion (of the paper by C. Terrell Bartlett, M. Am. Soc. C. E., published in November, 1921, *Proceedings*, and presented at the meeting of October 5th, 1921), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Boston, Mass.

‡ *Proceedings*, Am. Soc. C. E., November, 1921, p. 454.

mately 22 000 cu. ft. per sec., or nearly three times the flood flow in 1913. This figure seems to have been very nearly the maximum rate of flow in the river during the flood of September, 1921.

The flood of 1913 was so much in excess of any other flood within the memory of the people of San Antonio that provision for a 50% greater flow in the river channel probably seemed to them to be adequate for the protection of the city. If the recent flood had not occurred, local sentiment might not have followed the recommendations of the report to the extent of including the detention basin in the works to be constructed, although it is more or less probable that gradual improvement of the river channel to a capacity approximating that recommended in the report would have been carried out.

In view of the scarcity and unsatisfactory character of the information available, the conclusions of the report seem to be almost inspired prophecy, and it may be permissible to quote a few sentences showing how the conclusions have been promptly and disastrously verified by the recent flood. The report states:

"We doubt if the citizens realize the ruinous loss which would result to-day with the present condition of the river channels, from such a flood as that of a century ago (1819). When such a flood will recur, no man can say. But that it will recur is certain. * * * We counsel the wisdom of pushing this work, however, while the memory of recent floods is vivid, lest the public mind relapse into inaction in a false sense of security, only to be startled later by stern reality when the inevitable flood shall come. We urge that your citizens shall remember that this disastrous flood is just as likely to come next year as at any other time."

Remembering that this report was made in December, 1920, it appears that the prediction was verified within nine months of its presentation.

A report from the local Governor to the Provincial Governor, accounting for damages to Government property by the flood of 1819, appears in the county records. Texas was then a part of Mexico, which itself was a Spanish Colony. The property of certain rebels against the Government had been confiscated, including several houses of adobe construction. The flood water extended around some of these houses, which resulted in the softening of the walls so that the houses collapsed. It is impossible to say whether the water was 1 ft. or several feet deep, and there seems to be no information available regarding the condition of the stream channel. It is known that irrigation was practiced extensively, and it is possible that the diverting dams may have had considerable effect in raising the water surface. It is also probable that whatever bridges existed were of wooden trestle construction and may have seriously obstructed the flow of the stream. Exact knowledge of the flood height, therefore, would be insufficient as a basis for a satisfactory estimate of flood discharge. The report concluded that such discharge might have been between 20 000 and 30 000 cu. ft. per sec. Mr. Bartlett's conclusion is that the flood of 1819 was probably somewhat greater than that of 1921.

It is not impossible that records of the old Spanish Missions, the earliest of which was founded about 1720, might also contain information of great significance in this connection. The speaker, however, does not know whether such records exist.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

WILLIAM EDGAR BAKER, M. Am. Soc. C. E.*

DIED NOVEMBER 7TH, 1921.

William Edgar Baker was born on October 18th, 1856, at Springfield, Mass., in the home of his grandfather, General James Barnes. He was graduated from Lafayette College in 1877, with the degree of Civil Engineer.

Immediately following his graduation, Mr. Baker joined the Engineering Staff of the St. Paul and Pacific Railroad Company (now the Great Northern Railway Company) and served as Transitman and Engineer in charge of work until 1880.

He then entered the service of the Canadian Pacific Railroad Company as Transitman on surveys for that line through the Rocky Mountains. This work was conducted under such conditions of hardship that few of the members of the party survived, and Mr. Baker was soon in charge of the expedition. His diary written at this time has been given to the Engineers' Club of New York City. In 1883, he was appointed Resident Engineer in charge of bridges and buildings of the International and Great Northern Railway in Texas, which position he held until 1888.

In 1889, Mr. Baker began his career as an Electrical Engineer as Superintendent of Electric Car Service for the Thompson Houston Electric Company (now the General Electric Company), in Boston, Mass., where he had charge of the installation of the electric equipment of the West End Street Railway.

In 1892, he went to Chicago, Ill., where he designed and built the Columbian Intramural Railway at the World's Columbian Exposition, serving as General Manager of the railway until the close of the Exposition. This work was the first application of the third-rail system, and brought Mr. Baker into prominence as an Electrical Engineer both in the United States and in Europe.

The Metropolitan West Side Elevated Railroad, in Chicago, was under construction at that time, and although the Company had contracted for steam locomotives for use in its operation, Mr. Baker was asked to convert it into an electric road, using the third-rail system with which he had been so successful on the Intramural Railroad at the World's Fair.

In 1899, he came to New York City where he equipped the Manhattan Elevated Railroad with electricity. As General Manager of the Company, he operated the line until he resigned to open an office as a Consulting Engineer.

After engaging in private practice, Mr. Baker designed and supervised the construction of the electrical equipment of the Calumet and Hecla

* Memoir compiled from information furnished by W. E. Baker and Company and on file at the Headquarters of the Society.

Copper Mines. He constructed at Chicago the first bascule bridge over the Chicago River, and was also consulted in connection with the construction of the London Underground Railways. After retiring from active business, he spent much of his time at his country place at Chester, N. J., where he was interested in scientific agriculture.

Mr. Baker was married in 1884, to Miss Harriet Griffin, who, with five children and eight grandchildren, survives him.

He was a member of the Engineers' Club of New City and a Trustee of Lafayette College, in the welfare of which he was deeply interested.

Mr. Baker was elected a Member of the American Society of Civil Engineers on June 1st, 1898.

FREDERICK WILLIAM CAPPELEN, M. Am. Soc. C. E.*

DIED OCTOBER 16TH, 1921.

Frederick William Cappelen was born at Drammen, Norway, on October 21st, 1857. He received his high school education at Frederickstad, Norway, and was graduated from the Technical School of Oerebro, Sweden. He afterward attended the Polytechnicum at Dresden, Germany, from which he was graduated as a Civil Engineer with the highest honors ever won at that time by a foreign student at the school.

In 1880, Mr. Cappelen, came to the United States and was employed in the Engineering Offices of the Northern Pacific Railroad in New York City and at Brainerd, Minn. From 1881 to 1884, he served as Assistant Engineer on location and construction of the Missoula Division of the Northern Pacific Railroad, in Montana, and from 1884 to 1886, he was engaged, as Assistant Engineer, on bridge work with the St. Paul and Northern Pacific Railway Company.

He then entered the employ of the City of Minneapolis, Minn., as Bridge Engineer, remaining in that position until 1892. During this time, among other work, he designed and built three highway bridges across the Mississippi River.

On January 2d, 1893, Mr. Cappelen was elected City Engineer of Minneapolis and held that office until 1898. In 1913 he again became City Engineer, remaining in office until his death. During this latter term, he built the St. Anthony Bridge, which crosses the Mississippi at St. Anthony Falls. This bridge is a fine example of reinforced concrete construction, 2 223 ft. long and 80 ft. wide, and is conceded to be a noble and enduring structure.

The Franklin Avenue Bridge across the Mississippi, another reinforced concrete highway bridge, 1 030 ft. long with five spans, the central one of which is 400 ft.—one of the longest concrete spans in the world—was designed by Mr. Cappelen, and was in the process of erection under his direction at the time of his death.

* Memoir prepared by George L. Wilson, M. Am. Soc. C. E.

In 1895, he designed the Minneapolis Reservoir System which was the first step toward the purification of the water supply of that city. His interest in and designs for the improvement of the city water-works, caused him often to be referred to as the "Father" of the present system.

From 1898 to 1913, Mr. Cappelen maintained an office as a Consulting Engineer on bridge and municipal work. During this time, he had a large part in the bridging and lowering of the steam railroad tracks through Minneapolis, and also acted as Consulting Engineer for the Great Northern Railway Company.

In 1904, he served on a commission with Mr. Andrew Rinker, then City Engineer, and Allen Hazen, M. Am. Soc. C. E., of New York City, for the investigation of a pure water supply for Minneapolis, and their report recommended the purification of the present supply from the Mississippi River. In 1913 the operation of the filtration plant was begun under his direction, and in 1921, its capacity had been increased to 90 000 000 gal. per day. During the thirteen years in which Mr. Cappelen served Minneapolis as City Engineer, the water-works system was rapidly extended and improved, keeping pace with the growth of the city.

From 1907 to 1911, he was also connected with the Decaries Incinerator Company, working out, during this time, many improvements in its garbage reduction process.

Extensive studies and investigations were made by Mr. Cappelen on the subject of grade separation at street and railroad crossings and a part of this work has already been brought to a successful completion; general plans for much more were completed by him shortly before his death.

His standing and reputation as a Sanitary Engineer was recognized by the Governor of Minnesota, who, in 1918, appointed him as one of the first two engineer members of the State Board of Health, and in January, 1921, he was again appointed to this position.

He was married in 1883 to Miss Felicitas Wessel, of Dresden, Germany, who, with two sons, Arthur S., of New York City, and Felix G., of Akron, Ohio, survives him.

Mr. Cappelen was a member and Trustee of the American Water Works Association, a member of the American Society of Municipal Improvements and of the Minneapolis Engineers' Club. He was also a member of the Odin Club, of the Masonic Order, the Elks, and many civic organizations.

His place in the estimation of the public was well stated in a leading city paper on the day following his death:

"For a quarter of a century this man had given his best in public service to Minneapolis. He used his engineering genius, not merely to facilitate communication, but to preserve public health, to promote safety and to enhance the attractions of the city. His devotion to the duties of his office and to the upbuilding of Minneapolis was not without its personal sacrifices. His passing is a loss that will not easily be retrieved."

Mr. Cappelen was elected a Member of the American Society of Civil Engineers on April 3d, 1895.

PHILIP À MORLEY PARKER, M. Am. Soc. C. E.*

Died August 4th, 1920.

Philip À Morley Parker, the younger son of Captain Philip À Morley Parker, R. N., was born at Greenwich, Kent, England, on May 31st, 1872. He was educated at Cheltenham College and his great mathematical ability enabled him to reach the top of the mathematical side before he left. Choosing the Army as a profession, he passed into the Royal Military Academy, Woolwich, but was rejected owing to a weak heart by which he was handicapped throughout his life. In 1889, Mr. Parker entered the Central Technical College (now The City and Guilds of London Engineering College of the Imperial College of Science and Technology) and studied Civil and Mechanical Engineering under Professor Unwin, but on the appointment of his father to the Australian Squadron, he left England before he had completed the course. He continued his engineering studies at Melbourne University and took the Degree of Bachelor of Civil Engineering (B. C. E.) in 1894. On his return to England, he entered Cambridge University, took the Mathematical Tripos in 1897, was 12th Wrangler (notwithstanding illness during the examination), and Senior Scholar of St. John's College.

In May, 1898, Mr. Parker entered the office of Messrs. Hunter and Middleton, Civil Engineers, London, England, and at the end of his pupilage acted as Assistant Resident Engineer on the construction of the Staines Reservoirs—an extensive part of the London Water Supply.

After three years of independent consulting practice in London, he entered the Indian Irrigation Service in 1905, and during the following five years rendered valuable services with respect to design, original investigation, and in an executive capacity, as Assistant Engineer on the project for the Upper Swat Canal, Assistant in charge of the Lower Jhelum Canal Irrigation Area, and in charge of the Second Division of the Upper Chenab Canal. He also served as Executive Engineer in charge of the Upper Jhelum Canal and as Officer in charge of general hydraulic experiments, in pumping on the Upper Bari Doab Canal, and on weir and sluice discharges, etc.

Resigning from the Punjab Irrigation in 1910, Mr. Parker began his book, "The Control of Water", and in order to complete his study of irrigation and general hydraulic practice, he visited the United States, Japan, Ceylon, Egypt, and Italy. While occupied on his book and up to the beginning of the World War, he continued his consulting practice in London and, during that period, carried out some irrigation projects in Jamaica.

In the early stages of the World War, when the fulfillment of urgent requirements largely depended on the initiative of individuals, Mr. Parker did extremely valuable work. With several friends he gave lectures to a large number of Army Service Corps recruits on water supply in the field and practical demonstrations of the application of the best safeguards to neutralize

* Memoir prepared by J. S. Wilson, Esq., London, England.

harmful qualities in water. He also printed and distributed to officers and men, at his own expense, a vocabulary relating to the subject in English, French, and German, with which was incorporated some concise instructions. Early in 1915, before any Government departmental system had been set up to deal with water questions, Mr. Parker undertook to provide a water supply for a large plant which was being transformed into an explosives factory by Lord Moulton's newly formed committee. The plan which he prepared and carried out included the laying of an 18-in. water main, 8 miles long, from the Halkyn Mountains to Queensferry on the Dee. The active co-operation of a contractor and the use made by Mr. Parker in his executive capacity of the emergency powers to overcome obstacles, enabled him to complete the work in a phenomenally short time. In laying the pipe a maximum rate of 1 400 ft. per day was attained.

The winters proved trying to his health and he left London for the last time in May, 1916, to take the appointment of Chief on the Constructional Staff on the Sydney Underground City Railway. Work on this railway was discontinued in 1917, owing to a change of policy by the Government. From Sydney, Mr. Parker went to the Federated Malay States, and there irrigation projects in connection with land development proved to be his last work, for while thus engaged he died of heart failure at Kuala Lumpur, on August 4th, 1920.

By his death the Engineering Profession has lost one of its younger and more highly trained exponents. Mr. Parker's personality was an attractive one and, coupled with his great ability, gave him a large circle of friends; generally, however, he was best known as the auther of "The Control of Water" published in 1913. This work covers almost the whole science of Hydraulic Engineering, and Mr. Parker, in addition to a profound knowledge of the underlying principles, had the advantage of a wide practical experience, a combination which places the book in a unique position among manuals on "Hydraulics".

Throughout his career, Mr. Parker was a keen student of engineering, particularly of the branch he had made his own. He had carefully followed all original investigations and experiments reported from all parts of the world and, in his own work, whenever there was an opportunity of making an investigation, he made full use of his ability with the object of converting the results of his experience into rules or principles of general application, and in doing this he carefully considered any other work of importance bearing on the matter. In "The Control of Water", every subject throughout the wide range covered is reviewed and examined in a masterly manner and with a completeness of reference and cross-reference which makes the book not only the best in the language on the subject, but "a monumental contribution to applied hydraulics", as it was described in a review by an eminent American authority. Mr. Parker had other books in preparation at the time of his death.

Mr. Parker was elected an Associate Member of the American Society of Civil Engineers on March 6th, 1907, and a Member on January 6th, 1915. He was also a Member of the Institution of Civil Engineers.

WILLIAM HENRY SEARLES, M. Am. Soc. C. E.*

DIED APRIL 23D, 1921.

William Henry Searles, the son of Ashbury M. and Rachel Mitchell Searles, was born in Cincinnati, Ohio, on June 4th, 1837.

He attended private schools for preparatory work for Wesleyan University, at which he was a student in 1856 and 1857. Later, he entered Rensselaer Polytechnic Institute, Troy, N. Y., from which he was graduated in 1860.

In 1861, Mr. Searles served as Assistant Engineer of the Marietta and Cincinnati Railroad, and as Military Engineer on the staff of Gen. Buell and under Gen. Rosecrans, on the defenses of Cincinnati.

His interest in technical and scientific work led to his service as Professor of Geodesy and Topography at his Alma Mater in 1862 and 1864.

In 1865, he became Chief Engineer of the Mineral Range Railroad of Michigan, now a part of the Duluth, South Shore and Atlantic Railroad, and, in 1866, Assistant Engineer of the Allegheny River Bridge, for the Pittsburgh, Fort Wayne and Chicago Railroad Company. In 1867, Mr. Searles was employed as Principal Assistant Engineer of the Allegheny Valley Railroad.

During 1868-69, he was engaged as a manufacturer of petroleum products at Cleveland, Ohio, but in 1870-71, he again engaged in the practice of his profession as Chief Engineer of the Indiana North and South Railroad, at Indianapolis, Ind., and, in 1872-73, as Chief Engineer (7th Corps of the New York and West Shore Railroad, now a part of the New York Central System.

From 1876 to 1878, Mr. Searles acted as Consulting Engineer on the New York State Canals, following which, he entered private practice as a Consulting Engineer, in Cleveland, Ohio.

After retiring from active work, Mr. Searles took up his residence in Elyria, near Cleveland, Ohio, where he was actively engaged in civic and church affairs. For a time he was Superintendent of the Sunday School of the First Congregational Church which he also served as Clerk for thirteen years and as Clerk Emeritus until his death.

He was well known as the author of Searles' "Field Engineering" which was published in 1880, and of the "Railroad Spiral", which was issued in 1882.

From 1888 to 1890, Mr. Searles was a member of the Board of Managers of the Cleveland Engineering Society, in the work of which he took a deep interest. He was President of this Society in 1890 and 1891, and, later, in 1897, served as its Secretary. In 1907, in recognition of his engineering and scientific attainments, he was elected an Honorary Member.

He was married in 1870 to Mary L. Doolittle, who survives him, the fiftieth anniversary of their wedding having been observed by Mr. and Mrs. Searles and their many friends at Elyria about a year before his death, which occurred on April 23d, 1921.

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* Memoir prepared by W. P. Brown, Frank C. Osborn, and James Ritchie, Members, Am. Soc. C. E., a Committee of the Cleveland Section of the American Society of Civil Engineers.

Mr. Searles possessed the faculty of a thorough and correct analysis, as well as a clear and concise expression of his conclusions, and, in addition to his attainments in the Engineering Profession, he will be remembered as a Christian gentleman who always strived for high ideals in his professional and private affairs.

Mr. Searles was elected a Member of the American Society of Civil Engineers on July 2d, 1873.

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